PREDICTION AND EVALUATION OF A LATERALLY LOADED PILE GROUP

BY

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To my father

Armando Ruesta Ledesma

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PREDICTION AND EVALUATION OF A LATERALLY LOADED PILE GROUP

By

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An isolated single pile and a large scale test group of 16 prestressed concrete piles in a closely spaced arrangement were subjected to static lateral loading using a fixed-headed group for reaction. Ten piles of the test group, six piles of the reaction group, and the single pile were instrumented with strain gages and inclinometers. The piles were 30 inches in width and 54 feet long installed in a 3-d center to center spacing. The tests were conducted in cohesionless soil overlying partially cemented sand at Roosevelt bridge replacement, in Stuart, Florida

Details on the site investigations, soil properties, pile geometry, instrumentation, and loading test procedures are presented. Results of the load tests were compared with the program FLPIER (LPGSTAN) that considers both the nonlinear nature of the soil and the prestressed concrete to predict lateral pile behavior.

Different methods, based on instrumentation and in situ tests(prior and post pile driving), were used to calculate load-transfer curves (i.e. p-y and t-z curves). Some soil densification was observed due to soil displacement within the pile group according to in situ measurements (PMT, CPT, DMT, and SPT). The reduction of soil resistance due to group effect was evaluated by p-y multipliers in both pile groups

Finally, some recommendations for the evaluation of p-y curves and the design of closely spaced pile groups subjected to lateral loading are presented.

CHAPTER 1
INTRODUCTION

The Florida Department of Transportation is constructing a bridge structure as a replacement to the existing Roosevelt draw-bridge over the St Lucie River in Stuart, Florida This bridge will span approximately 3500 feet over the river with a total length of 4600 feet adjoining U S. highway 1.

A full scale instrumented free-headed laterally loaded test of sixteen prestressed sacrificial piles and an isolated pile test were performed using a bridge pier (Pier 16 N) for a reaction system

1 1 Problem Statement

Lateral loading design considerations are generally of primary importance for deep foundations subjected to forces such as ship impact, wind, earth pressure, water pressures or earthquakes Because of the fact that few full-scale lateral load group tests of deep foundations have been performed due to cost limitations, there is a lack of clear design guidelines for pile groups. When the lateral load control the design, this may cause overconservative designs, as it is in Roosevelt bridge according to the design phase pile load test program from Law Engineering (1992).

For a single pile, lateral loading is a problem of soil-structure interaction, so that deflection of the pile depends on the soil response and the soil response depends on the pile deflection (Reese, 1984). For a closely spaced pile-group this behavior is more complex due to two stress conditions, defined in a near-field zone of high stresses, nonlinear soil behavior (large displacements in a region close to the pile), and in a far-field zone of relatively small stresses and strains between pile and pile (Ooi and Duncan, 1994).

Despite the significance of closely spaced pile interaction, there exists a lack of knowledge concerning pile group effects. Most lateral load investigations have been done on isolated single piles, even though piles are most frequently used in groups.

1.2 Background

Currently, to solve the lateral load pile group problem there are three major methodologies: numerical modeling, small scale laboratory models, and full scale load testing. Numerical modeling is based on finite elements or finite difference methods, but its accuracy is limited mainly to the determination of soil or pile properties Small scale models, i e centrifuge tests, are a technique that simulate stress-strain conditions observed in the field The models can be used to evaluate/verify numerical codes, but modeling difficulties limit site-specific field predictions.

Current analysis trends are using theories that account for the nonlinear behavior of soil depending on soil type, loading and pile geometry A widely acceptable solution for lateral load design in piles is the p-y approach, that is, a Winkler or a subgrade reaction approach which utilizes a beam-column on an elastic foundation with nonlinear springs to transfer the load from the pile to the soil. These springs represent the total soil resistance at a particular depth to the lateral displacement of a horizontally loaded pile. Recommendations for the prediction of p-y curves are based on the results of full-scale tests where the continuum effect is explicitly satisfied There is a relative close agreement between results obtained from experiments and theoretical solutions.

For closely spaced pile groups, only a few full-scale instrumented tests have been performed and most of them were short-term static load tests that were used as a reference to correlate to other types of loading A well-known fully instrumented pile load test was described by Brown et al (1988). This test was performed using a 3 by 3 steel-pipe pile group. The results were fairly good; however, each pile load test is unique due to the variable nature of ground and particular test conditions. Therefore, it is necessary to obtain more data to establish/validate a reliable method of design for pile groups with a better knowledge of the materials involved. As part of Florida DOT research, the University of Florida geotechnical group is performing centrifuge tests of different pile group configurations to determine p-y multipliers for different pile spacing, soil conditions, etc.

1.3 Objectives

There are four major objectives of this research.

1) Provide a better understanding of the lateral resistance of closely spaced (3-d) driven piles in a group and whether this can be related to the behavior of a single isolated pile through p-y multipliers.

2) Evaluate techniques for determining p-y curves based upon in situ tests and the estimation of soil densification due to jetting and driving in a pile group.

3) Verify computer programs that are widely used by FDOT for design such as FLPIER (latest version of LPGSTAN). The prediction was compared with the actual behavior obtained from the laterally loaded pile tests considering the nonlinear

nature of soil and the nonlinear behavior of prestressed concrete piles. Also, the behavior of the piles within the reaction pier was evaluated considering the pile cap effects.
4) Provide a general guideline for future load tests and design recommendations for piles subjected to lateral loads for more realistic and economic projects

1.4 Scope of Work

To accomplish these objectives, the following tasks were performed: a) in situ soil testing, b) instrumentation and data reduction, c) static lateral load tests, d) p-y curves, and, e) prediction and evaluation of results.

a) In situ Soil Testing

In addition to FDOT's site SPT N values the University of Florida has performed cone penetration (CPT), dilatometer (DMT), and cone-pushed pressuremeter tests (CPMT). Preliminary in situ tests were performed adjacent to Pier 19 and the north abutment with the UF cone truck and within Pier 18 using a modified Cambridge self-boring pressuremeter trailer. These tests were performed to determine if a hard layer of cemented sand could be penetrate and to prove the equipment and methodology.

To investigate if soil densification occurs due to pile driving, pre-driving in situ soil testing was performed in the vicinity of Pier 16 and post-driving testing was perform at Pier 16 and between the piles in the test group using the UF cone truck and the trailer.

b) Instrumentation and Data Reduction

The instrumentation mainly consisted of strain gages, load cells, manual and automatic slope inclinometers, wire-lines and potentiometers. The "fixed" group (Pier 16 North), had 6 piles fully instrumented (inclinometer and 7 levels of strain gages) for axial and bending strains, and the free headed test group had 10 piles fully instrumented (inclinometer and 9 levels of strain gages) for bending strains only. Additionally, a fully instrumented (strain gages and inclinometer) single isolated pile was tested.

c) Static Lateral Load Tests

A single isolated pile was laterally loaded to 72 kips yielding and the lateral pile test group to a maximum lateral load of 1080 kips, i.e. close to the test group yielding. Sixteen 30" piles connected through the cap were the reaction pier ("fixed") for the 16 sacrificial piles ("pinned") joined by a steel frame at Pier 16 North.

d) P-Y Curves

The deflection and soil reaction for the p-y curves were determined by a family of bending moments obtained with the strain gages from the load tests. Even though it is not exact, it is the best known technique to determine p-y curves

Inclinometer readings were used as a complement to strain gage measurements to determine pile rotations and deflections as a function of depth. Also, a method to determine p-y curves directly from inclinometer readings is presented.

FLPIER/COM624P default p-y curves for cohesionless soils were compared with p-y curves obtained from methods based on in situ tests such as PMT and DMT. Finally, p-y curves derived from CPT and SPT based on the lateral load tests performed and practical suggestions are presented.

e) Prediction and Evaluation of Results

The deflections, loads, shear forces, and bending moments were determined mainly with FLPIER, considering the pile non-linear behavior for prestressed concrete. The p-y curves previously obtained were used to predict the behavior of the single pile, the reaction pier (I6 N), and the sacrificial pile test group. These predictions were compared to the load test results.

Single pile behavior also was predicted using FLPIER with linear pile properties, COM624P, and PIGR3D without pile interaction. For the pile groups response, FLPIER was compared with GROUP and PIGR3D with interaction. FLPIER and GROUP used p-y multipliers as a function of row position to model pile group effects, while PIGR3D used Mindlin's solution to analyze pile-soil-pile interaction.

CHAPTER 2

LITERATURE REVIEW

The purpose of this chapter is to provide a review of the theory and software for predicting lateral behavior of single isolated piles and closely spaced pile groups.

2.1 Behavior of Single Piles

Lateral loading of a single pile is a problem of soil-structure interaction, in that deflection of the pile depends on the soil response and the soil response depends on the pile deflection. Three common approaches are used in the analysis of single piles.

a) Model the soil as a series of closely-spaced discrete springs with no coupling between them. This is known as the unit load transfer model or Winkler approach.

b) Model the pile as a beam on an ideal elastic continuum (Poulos, 1971).

c) Model the soil using the finite element approach.

The most common solution for lateral load design of piles is the p-y approach; i.e. the Winkler assumption, due to the fact that it is simple to use and easy to modify for nonlinear soil response and cyclic loading. Elastic continuum models are used frequently for problems with small soil strains, such as dynamically loaded foundations. According to Poulos (1971), an elastic homogeneous isotropic soil better approximates a real soil mass, particularly for clayey soils than a Winkler medium. Finite element techniques remain as research tools due to considerable human and computer effort.

The Winkler or subgrade reaction approach utilizes a beam-column on an elastic foundation with nonlinear springs to transfer the load from the pile to the soil. These springs model the relationship of soil resistance versus deflection (p-y curves) that represents the total resistance at a particular depth to the lateral displacement of a horizontally loaded pile. Soil resistance will reach a limit and then remain constant or decrease with further deflection. This nonlinear behavior is opposite to pile deflection. The soil modulus E_s is stiffer at smaller deflections and decreases as the deflection of the pile increases as shown in Figure 2.1. The Winkler solution utilizes the beam theory with an axial load and horizontal springs that use a soil modulus E_s times pile deflection as force per length. This solution utilizes Hetenyi's beam-column theory that accounts for the axial load. This problem cannot be solved by static equilibrium, but is a fourth-order differential equation for elastic deflection of a beam that must be solved by iteration. Assuming equilibrium of moments and ignoring second-order terms, it leads to equation 2.1 that is a general solution for inelastic pile rigidity as it is considered in COM624P.

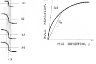


Figure 2.1 Typical P-y Curve (Reese, 1984)

$$\frac{d^2}{dx^2}\left(E\,I\,\frac{d^2y}{dx^2}\right) + P_x\,\frac{d^2y}{dx^2} - p - W = 0 \tag{2.1}$$

where

y is the deflection of the pile along the depth x,

EI is the rigidity of the pile,

P_x is the axial load on a pile,

p is the soil reaction per unit length,

W is the distributed load along the pile.

The solution of the differential equation is usually obtained by finite difference techniques, a well-established procedure in geotechnical engineering. This solution gives values of y as a function of x and a family of curves, where EI varies along the pile. The shapes from a complete solution are shown in Figure 2.2, where

y (x) is the deflection along the pile

y' is the slope of the pile $\quad y' = \frac{dy}{dx}$

M is the bending moment $\quad M = \frac{d\ (E\ I\ y)}{dx^2}$

V is the shear $\quad V = \frac{d}{dx^2}\ (E\ I\ \frac{dy}{dx})$

P is the soil reaction $\quad P = \frac{d}{dx^2}\ (E\ I\ \frac{d^2y}{dx^2})$

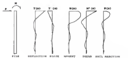


Figure 2.2 Complete Solution of Differential Equation

2.2 Behavior of Pile Groups

Although piles are frequently used in group, there exists a lack of knowledge concerning pile group interaction. For a closely spaced pile group, lateral behavior is more complex due to two stress conditions, 1) a near field zone of high stresses and plastic strains (large displacements in a region close to the pile), and 2) a far field zone of relatively small stresses and strains between pile and pile (Ooi and Duncan, 1994). An additional boundary complexity is that piles are embedded into a monolithic reinforced concrete cap. This leads to another form of influence in pile group behavior, that is, the interaction between pile tops which are connected to a pile cap sharing forces from the superstructure. In most of the cases an assumption of a complete fixity of the pile cap is accepted.

In order to analyze closely spaced grouped piles, various analytical models have been developed by making assumptions that handle the problem. The most common methods are as follows:

a) Finite element model
b) Continuum model
c) Modified continuum model
d) Single pile method
e) Modified unit load transfer

f) Hybrid model

The finite element model could be used as a powerful tool to represent nonlinear soil behavior and different strata. However, it requires knowing appropriate constitutive equations, soil parameters and the initial state of stress surrounding the piles.

The continuum model is based on solutions presented by Poulos (1971) to calculate the behavior of a pile group under lateral loading. The model consists of an elastic half space and uses Mindlin's three-dimensional equations to model pile-soil-pile interaction. It has the disadvantage of a large computational effort and the difficulty of establishing appropriate elastic parameters.

The modified continuum model (or coupled Winkler model) is the continuum model with the exception that the pile-soil-pile interactions are assumed to occur only in horizontal planes in order to reduce computational effort.

The single equivalent pile method was proposed by Reese (1984) to establish an upper bound on pile group deflection and bending moments. It assumes that the soil contained within the piles moves with the group. Thus, the piles and the enclosed soil are treated as a single pile of an imaginary large diameter. This diameter is taken as the perimeter of the group divided by π. The total stiffness is determined as the sum of individual pile stiffness, neglecting soil stiffness.

Shear and moments are distributed equally among the piles in the group

The Matlock-Bogard modified unit load transfer model cited by Morrison and Reese (1988) combined the behavior of an isolated pile with the behavior of the large imaginary pile in an empirical but rational method. They modified the p-y curves for the imaginary pile, by dividing the soil resistance, p, by the number of piles and the deflection, y, by the pile spacing in diameters. The deflection of the resultant p-y curve is the sum of the isolated and the modified deflections for the corresponding soil resistance

The hybrid model was initially proposed by Focht and Koch (1973) and is a combination of the continuum model and the unit load transfer model. They postulated that the pile group behavior is controlled by two different stress conditions; a zone close to the pile of high stresses which would result in large displacements and plastic strains and a zone between piles of relatively small stresses and strains. The p-y method with nonlinear curves is used for modeling pile-soil interaction and the additional displacements due to pile-soil-pile interaction are calculated using an elastic half space analysis proposed by Poulos. Finally, group deformation is predicted by assuming superposition of computed displacements. An extension and more refined approach was proposed by Ha (O'Neill et al., 1977), using the superposition concept. They hypothesized that each individual pile in the group was modelled by the use of nonlinear unit load transfer curves (f-w, q-w and p-y curves) and the interaction among piles was represented by means of elastic half space deformation theory. The piles were discretized and a finite element method was used representing each discrete element as a Mindlin point load. The deflection of each p-y curve is modified by the effect of the other piles and the response of each pile is recomputed using the modified p-y curve until it converges. This procedure produces a unique solution and is much more general than that of Poch and Koch (1973). Unfortunately it is very time consuming to execute on a digital computer because the individual interaction between every node on every pile in the group is computed and the solution sometimes becomes unstable.

Research is continuing on developing qualitative solutions from full scale experiments that will predict the influence of pile spacing on pile-soil-pile interaction. No definitive conclusions have been reached but a procedure that appears to be useful is to reduce soil resistance (p) and/or expand the deflection (y) from the p-y curves of an isolated single pile; i.e. each p-y curve is individually modified for the effect of the other piles accounting pile-soil-pile interaction. Brown et al. (1988) proposed experimental factors (p-y multipliers) that depend on row position and soil type in order to account for the 'group effect' that would represent the loss in soil resistance for a group of piles compared to an isolated pile. The graphical concept is presented in Figure 2.3. This concept is employed in Florida-Pier.

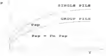


Figure 2.3 P-y Multipliers Concept (Brown et al., 1988)

Centrifuge tests were performed at the University of Florida on a single pile and on a 3x3 pile groups at 3-d and 5-d spacings (McVay et al. 1995). The piles were driven in flight to simulate geostatic stresses during driving. These tests were performed in two different relative soil densities, medium loose (Dr=33%) and medium dense sand (Dr=55%). The shadowing effect was reported with a good agreement with the Texas pile group. For the 3-d spaced group (Roosevelt bridge pile spacing) the p-y multipliers were 0.8, 0.4, and 0.3 for very dense to medium dense sand, and 0.65, 0.45, and 0.35 for medium loose sands. The group efficiency was 0.74 and the load distribution varies from a high of 45%, 32%, 23% for very dense to 37%, 33%, 30% for medium loose sands.

The latest research involved centrifuge testing of large fixed head lateral groups (3x3 to 3x7 rows of piles) in medium loose (Dr=36%) and medium dense sand (Dr=55%) at 3-d pile spacing (McVay et al., 1996). Also two single pile load test were performed, the tests were repeated twice for each combination. They stated that the individual row distribution is independent of the group size, but is dependent on row position. The latter supports the p-y multiplier concept. The p-y multipliers were found to be 0.8, 0.4, 0.3, 0.2, and 0.2 for all densities.

2.3 General Comments on Programs to be Used

Some assumptions, comments, and capabilities of common programs to predict lateral pile behavior are presented.

2.3.1 COM624P (v2.0)

COM624P uses the Winkler foundation assumption; thus, it solves the differential equation 2.1 based on beam column theory. It does not consider group effect limiting the problem to an isolated single pile. The only adjustment is to replace the distribution load p with the secant modulus of soil reaction (E_s) times pile deflection (Y), with a negative sign. The term W is similar to p and is added to allow a distributed load along the pile i.e. when the pile extends above the mudline.

Soil response is characterized as a set of discrete mechanisms such that the soil resistance at a point is not dependent on pile deflection elsewhere, thus, a continuum is not perfectly modeled. However, the recommendations for predicting p-y curves are based on field experiments in which the continuum effect was explicitly implemented. This method is very versatile in dealing with the highly nonlinear soil-pile interaction system with any arbitrary change in soil and pile properties. The program uses a finite difference method to solve differential equation 2.1. The pile is discretized to use the recurrence formula in equation 2.2. A graphical representation of the discretized pile is shown in Figure 2.4.

$$\begin{aligned} & R_{i-1}\, Y_{i-2} + [P_x\, h^2 - 2R_i - 2R_{i-1}]\, Y_{i-1} + \\ & [R_{i+1} + 4R_i + R_{i-1} + E_{si}\, h^4 - 2P_x\, h^2]\, Y_i + \\ & [P_x\, h^2 - 2R_{i+1} - 2R_i]\, Y_{i+1} + R_{i+1}\, Y_{i+2} = 0 \end{aligned} \qquad (2.2)$$

where

R_i is the Flexural rigidity at point i ($R_i = E_i\, I_i$).

The deflection at any point can be known by applying some boundary conditions to the top and bottom of the pile. Bending moment and shear force at node 0 (bottom) are zero. Lateral force, moment, and slope of the elastic curve or rotational restraint constant (Mt/St) are known at the top of the pile.

Finally, a global matrix for the pile is assembled and the following equation in matrix form is solved:

$$\{P\}=[K]\cdot\{U\} \qquad (2.3)$$

Where

{P} represents external forces,

[K] is the rigidity matrix, and

{U} are the deflections at every node.

As p-y curves may be nonlinear, E_s varies with y, an iterative process must be employed to solve until deflection assumed is similar to deflection computed.

COM624P (1993) has five default options to generate internally p-y curves for different soils conditions: soft clay below watertable, stiff clay below watertable, stiff clay above watertable, sand above and below watertable, limestone using unconfined compression strength.

An option for linear interpolation of user defined p-y curves is also available. However, this option can not be mixed with any default p-y curves. P-y curves can be modified according to the nature of loading (static or cyclic).

COM624P main program allows the user access four choices: file, input, run, and graphics. For each loading condition, COM624P gives a complete solution of equation 2.1, deflection, moment, total stress, shear, soil resistance, and flexural rigidity. As the flexural rigidity of reinforced concrete varies nonlinearly, the program considers a variation of EI when cracking occurs assuming that a constant EI may result in some degree of inaccuracy (COM624P, 1993). However, this option only works for circular sections.

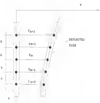


Figure 2.4 Representation of Deflected Pile (Reese et al., 1990)

2.3.2 FLORIDA-PIER

FLORIDA-PIER is a nonlinear finite element program developed at the University of Florida for analyzing bridge pier structures (piles or shafts, pile cap, pier columns, and pier cap, see Figure 2.5). The program models reinforced or prestressed concrete piles after cracking and soil nonlinear behavior. It is a flexible program that can model different pile and pier configurations, i.e., battered piles, variable spacing between piles or shafts and allows missing piles. FLPIER has a graphical pre-processor PIERGEN that can be used to create input files and a post-processor PIERPLOT to obtain deflected shapes, nodal information, deformations, rotations, internal stresses, moments, and forces in members.

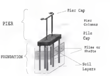


Figure 2.5 Bridge Pier Structure Unit (McVay et al., 1996a)

This program is the latest version of LPGSTAN that merged STAN (standard structural analysis) with LPG (axial and lateral pile loading). FLPIER models the piles, pier columns, beams, and pier cap with three dimensional beam elements. Each pile is generated with sixteen two-node beam elements with six degrees of freedom per node. Three degrees of translation and three rotations at each node. The element models bending in both planes, torsion and axial effects. Generally, the piles have a free standing length above the mudline and equally divided length for the remaining portion of the pile. The pile cap is considered flexible and is modelled as a nine-node shell element, where pile positions make up the four corners of each element. Pier columns, pier cap, and extra beams are model with six degree freedom beam elements. The program considers stiff shell elements to connect the pile cap to the piles and the pier. The deck is considered by adding springs on the pier. An iterative secant method is applied for solution (McVay et al. 1996a).

Axial soil interaction is modelled with nonlinear soil springs, t-z curves. The axial soil resistance along the length of the pile and the soil tip model are given by McVay el al. (1989) equations. T-z curves correlations from in situ tests are presented by Casper (1995).

Lateral pile-soil interaction is modelled by nonlinear springs (p-y curves) at each node in the pile. In this program

O'Neill's p-y curves for cohesionless soil and cohesive soil are included besides default p-y curves presented in COM624P for sand and clay. Additionally, a user defined p-y curves option for linear interpolation is included for lateral soil response that can be combined with any default option (McVay et al. 1996a). Pile-soil-pile interaction can be modelled in two ways, with p-y multipliers or with linear elastic springs. The latter considers soil between piles as a linear elastic medium interconnecting the piles. However, it is very computational expensive and is not recommended for personal computers. Thus p-y multipliers are usually used to account for overlapping shear zones. Experimental values of p-y multipliers from centrifuge tests have been back calculated for different pile group configurations (McVay et al. 1996b).

Earlier versions used a nonlinear model for soil but modelled the reinforced and prestressed concrete piles as linear elastic materials without considering cracking effects of reinforcement and prestressing. This overestimates the stiffness of a pile subjected to relatively high loadings in which the pile rigidity decreases after cracking.

FLPIER nonlinear pile response is described by Andrade (1994) and McVay et al. (1996b). The nonlinearity occurs primarily when the pile reaches the maximum bending capacity, after significant cracking, and the P-Δ moments become a significant portion of the bending capacity. FLPIER models material and geometric nonlinearities of the piles by 16 discrete elements with 12 degrees of freedom. The nonlinear material behavior is modeled by user input or default stress-strain curves for concrete, mild steel reinforcement, and prestressing steel strands. These stresses are integrated over the cross section of the piles by Gaussian Quadrature. The program considers the tension resistance of the concrete as 7.5 $\sqrt{f'_c}$ and the compression portion is defined by the Hognestad's parabola and a straight line. Figure 2.6 shows the default stress-strain curve for concrete. The prestressing concrete stress is the prestressing steel force divided by the concrete area. A prestressing stress-strain curve for f_{su} = 270 ksi is shown in Figure 2.7. For mild steel the curve is very similar but with a definite yield point usually at f'_y =60 ksi.

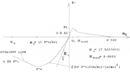


Figure 2.6 Stress-Strain Curve for Concrete

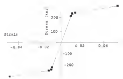


Figure 2.7 Stress-Strain Curve for Prestressing Steel

The nonlinear geometric behavior of the pile is modeled by considering the P-Δ effect i.e. the moments of axial force times the displacement of one end to another and the p-y effects i.e. the moments of the axial force times the displacements within members due to bending.

2.3.3 FIGR3D

This program was developed by Geosoft (Lazaridis,1993) to analyze the static response of a three-dimensional pile group under generalized loading with a nonlinear soil response and linear pile-soil-pile interaction, i.e. a hybrid method. Piles can be in any three-dimensional configuration and loads can be applied at any location. Pile head connections to the pile cap can be considered completely fixed or pinned. The pile cap is assumed to be rigid and does not interact with the soil.

Individual pile behavior is modeled by nonlinear unit load transfer curves, i.e. hyperbolic f-w (t-z), q-w and p-y curves. Soil characterization is limited for simplicity to three parameters: E_{sm} (initial soil modulus in psi), shape factor (usually 1), and f_u (maximum soil reaction in psi). No coupling among axial, lateral and torsional behavior for a single pile is considered and no group effect is generated by torsion.

The program employs finite difference and hyperbolic curves in order to generate six mode curves that relate deformations to forces and moments at pile heads. Then, it forms a stiffness matrix for each pile head and the stiffness matrix for the cap. Deformations of the group cap and at the pile head must be compatible. The process is repeated until convergence is achieved. The structure under an external generalized loading condition is in equilibrium with the reactions from every pile head and compatibility is maintained between the deformations and the reactions at the head of each pile. When pile-soil-pile interaction is taken into consideration a typical pile is chosen to reduce computational effort. Mindlin's equations are employed to calculate additional displacements due to interaction at every node of the typical pile. With the calculated displacements, new modified mode curves are generated and the entire process is repeated until convergence.

2.3.4 GROUP

This program was developed by Ensoft (Reese et al.,1990) to analyze the behavior of a two-dimensional pile foundation with arbitrary arranged piles. The program is directed principally at the case where the individual piles are so widely spaced that there is no influence of one pile on another. However, p-y multipliers can be used to predict the effect of pile-soil-pile interaction.

The soil near the ground surface principally determines lateral response and soil at depth determines axial response. According to Reese et al. (1990), the behavior under axial load and under lateral load are independent. The relationship between axial load and displacement are not affected by the presence of lateral deflection and vice versa.

The program merges the structural theory for a pile cap with laterally and axially loaded pile solutions. A two-dimensional arrangement and nondeformability of the pile cap are assumed to reduce the number of variables. The line of action of the resultant external load may be inclined or may be assume at any arbitrary position with respect to the structure. There are three conceivable cases of pile connection to the pile cap: pin connection, fixed head pile, and elastically restrained pile.

The structural theory for the pile group uses a numerical method to seek compatible displacement of the pile cap which satisfies the equilibrium of the applied external loads and the nonlinear pile reactions. A successive-displacement-correction method for obtaining the equilibrium state of the applied loads and the pile reactions on the pile cap has been developed. The solution is obtained through an iterative numerical procedure. Pile cap displacements are transformed to pile top displacements and these to pile reactions. The sum of pile reactions must be similar to the applied loads and the displacements at the pile cap and pile tops must be compatible.

The analysis of the laterally loaded pile is made by the finite difference method using a discretized beam-column in a similar way to COM624P. For axially loaded pile response, the program employs finite difference equations to achieve compatibility between pile displacement and resistance and displacements at pile tip and between load transfer curves along the pile. Additionally, a load-settlement curve can be used instead of the load transfer method.

2.4 Load Transfer Curves from Instrumentation

As previously discussed, the use of unit load transfer curves i.e. t-z and p-y curves appears to be a versatile and practical approach to represent axial and lateral nonlinear behavior of an individual pile. Most analyses assume that axial and lateral behavior are independent of one another. That is, the relationship between lateral load to lateral displacement is not affected by axial deflection and vice versa. Transfer curves obtained from instrumentation are considered as "measured curves" for this dissertation. The review presented herein is only related to cohesionless soils.

2.4.1 P-y Curves Based on Strain Gage Measurements

P-y curves can be obtained by numerical integration and differentiation of the bending moment function along the pile. To compute the 'measured' bending moments, before and after cracking, the measured strains must be related to the curvatures obtained using the nonlinear response from FLPIER. For a known cross section, the curvatures are calculated using a range of working moments at the top of a 'cantiliver' pile.

The measured bending moments along the pile can be fitted to polynomial equations using the least-square polynomial regression method. Deflection y(x) was obtained by double integration of the bending moment curves. The measured displacements and rotations at the pile head were used as constants of integration, sometimes they were complemented with inclinometer readings. The soil reaction p(x) was determined by double differentiation of the moment curve M(x). The resulting p(x) depends on the fitting curve and the number of points used. The order of polynomial to be fit must be less than the input number of data points.

The integration process usually can be solved without difficulties with the boundary conditions measured but the differentiation can lead to serious errors unless very accurate bending moments are measured. However, curve fitting can give acceptable results (Reese, 1979).

A family of bending moments was obtained for every instrumented pile during the load tests; bending moments are considered proportional to measured strains and pile rigidity.

2.4.2 P-y curves Based on Inclinometer Readings

Inclinometer measurements are used to determine pile rotations and deflections as a function of depth for every load increment. They also can be used as a complement of strain gage measurements to calculate the constants of integration for the p-y curves. A third degree differentiation can lead to serious inaccuracies when calculating soil resistance of p-y curves from inclinometer rotations.

A method that utilizes equation 2.3 was implemented in a spreadsheet to obtain p-y curves. The procedure follows the COM624P finite difference solution dividing the pile in equal segments, applying boundary conditions at the top and at the bottom of the pile, for the case when the lateral load F_t and moment M_t at the top of the pile are known. Soil modulus is modified for each segment until the actual pile deflection is as close as possible to the deflection obtained from inclinometer readings.

For cohesionless soils, the inclinometer readings also can be used together with the non-dimensional coefficients method presented by Matlock and Reese (1961) to calculate p-y curves. This method assumes that the soil modulus is a function of depth and the pile behaves as a long pile, i.e., the relative stiffness factor T is greater than four. In most of the cases a simple linear variation is used

$$T^5 = \frac{E\,I}{k} \tag{2.4}$$

$$y = A_y\,\frac{F_t\,T^3}{E\,I} + B_y\,\frac{M_t\,T^2}{E\,I} \tag{2.5}$$

The non-dimensional coefficients due to shear and moment are functions only of a depth coefficient Z which is equal to x/T. These coefficients for E_s proportional to depth are available (Matlock and Reese, 1960).

2.4.3 T-z Curves based on Strain Gages Measurements

The axial load transferred to the soil also can be represented by a series of mechanisms along any pile. Load-distribution curves along pile depth can be obtained from an instrumented pile under axial loads. The slope of these curves divided by the perimeter of the pile produces the unit load transfer at a certain depth i.e. discretizing the pile in segments is the difference in axial load between the top and the bottom of the segment divided by the surface of the shaft. The deformation is the strain times the segment length (Δx) i.e. the compression of the pile subtracted from the top movement. Repeating computations for each load produce curves that represent the load transferred to the soil versus vertical movement at certain points along the pile. Load and deformation are proportional to AE as shown in equation 2.6. These transfer curves are commonly presented as t-z curves, where t is the ratio of the load transfer to shear strength.

$$F = (A\,E)\,\varepsilon \qquad (2.6)$$

$$A\,E = A_s\,E_s + A_c\,E_c \qquad (2.7)$$

where

E_s and E_c are the moduli of elasticity of steel and concrete respectively.

2.5 Load Transfer Curves Based on In Situ Tests

In situ tests can be used directly or indirectly to determinate more accurate load transfer curves. P-y curves can be estimated indirectly through standard penetration tests (SPT) and cone penetration tests (CPT) and directly through Marchetti dilatometer tests (DMT) and pressuremeter tests (PMT). According to Campanella et al. (1989), the latter can provide better predictions of lateral pile response. SPT and CPT data as a direct approach can make only a partial contribution to lateral pile behavior. Perhaps, CPT can be related to the pressuremeter test (PMT) which involves enlarging a cylindrical cavity (Schmertmann, 1978).

Robertson et al. (1985) and Gabr and Borden (1988) proposed different methods to calculate p-y curves from the DMT; both methods calculate the ultimate lateral resistance from the internal friction angle ϕ. However, Gabr and Borden (1988) recommend the CPT method to calculate ϕ in conjunction with his method because Schmertmann's DMT procedure to determine ϕ seems to be very high.

In the PMT the soil deforms in a radial direction while in a laterally loaded pile, the soil in the back face will tend to follow the pile and only in the front face the soil deforms similarly to the PMT; it is reasonable to suppose that the geometric form of the expansion curve obtained from the PMT would be similar to the p-y curve acting on the front face of the pile. The uniform radial expansion in the PMT theory then needs to be related to the one-sided cavity expansion due to the lateral moving pile. Several methods have been proposed for the design of laterally loaded piles using pressuremeter data. Most of these methods used the pre-bored pressuremeter that does not model the disturbance caused during pile installation. Robertson et al. (1985) used the results from a full-displacement pressuremeter to define nonlinear p-y curves. According to their experience, the results provide an excellent comparison with lateral pile load test measurements.

The most important parameters that determine the p-y curve geometric form and magnitude in cohesionless soils are the soil modulus of subgrade reaction (that is related to the initial slope of the curve) and the ultimate soil resistance.

2.5.1 Reese, Cox, and Koop Method

Reese et al. (1984) performed a series of lateral load tests on steel piles. These tests were conducted in sand below the water table at Mustang Island, Texas, in 1974. They suggested a well known procedure to compute p-y curves on sand (COM624P). The curve is defined based on pile diameter, soil modulus and ultimate soil resistance.

The ultimate soil resistance must be the smaller of the given equations:

Passive wedge type failure equation:

$$p_{st} = \gamma x \left[\frac{K_o x \tan\phi \sin\beta}{\tan(\beta-\phi)\cos\alpha} + \frac{\tan\beta}{\tan(\beta-\phi)} (b + x \tan\beta \tan\alpha) + K_o x \tan\beta (\tan\phi \sin\beta - \tan\alpha) - K_a b \right] \tag{2.8}$$

Lateral flow around type of failure equation:

$$p_{sd} = K_a b \gamma x (\tan^8\beta - 1) + K_o b \gamma x \tan\phi \tan^4\beta \tag{2.9}$$

where

$\alpha = \phi/2$, $\beta = 45+\phi/2$, $Ko=0.4$, and $Ka=\tan^2(45-\phi/2)$.

The initial portion of the curve is defined by the soil modulus K (pci). For submerged sand according to its density they suggested 20 pci, 60 pci, and 125 pci for loose, medium, and dense sand respectively. Unit weight and internal friction angle should be modified for sand above the water table

2.5.2 O'Neill and Murchison Method

O'Neill and Murchison (1983) proposed a hyperbolic tangent function to describe the p-y curves in sand. The equation is given by

$$P = \eta \, A \, P_u \tanh \left[\left(\frac{k \, x}{A \, \eta \, P_u} \right) y \right] \tag{2.10}$$

$$A = 3 - 0.8 \frac{x}{D} \geq 0.9 \tag{2.11}$$

where

η is a pile shape factor (1 for prismatic piles), and

A is an empirical static loading factor.

The ultimate soil resistance must be the lesser value of the given equations:

$$p_u = \gamma \, x \, [b \, (K_p - K_a) + x \, K_p \tan \phi \tan \beta \,] \tag{2.12}$$

$$p_u = b \, \gamma \, x \, [\, K_p^3 + 2 \, K_o \, K_p^2 \tan \phi + \tan \phi - K_a] \tag{2.13}$$

The characteristic shapes of the p-y curves computed with Reese and O'Neill methods are very similar, the major difference is in the computation of the ultimate soil resistance.

2.5.3 P-y Curves Based on DMT Methods

Robertson et al. (1989) described a method for estimating p-y curves for displacement piles directly from the DMT. They employed a Matlock cubic parabola to predict P-y curves, which has the form:

$$\frac{P}{P_u} = 0.5\left(\frac{y}{y_c}\right)^{0.33} \qquad (2.14)$$

where

P is lateral soil resistance,

P_u is the ultimate lateral soil resistance (the lesser value given by equations 2.12 and 2.13),

y is the lateral pile deflection, and

y_c is the reference pile deflection.

The reference pile deflection for cohesionless soils is

$$y_c = \frac{1.642 \sin\phi\, \sigma_v}{E_D\, F_\phi\, (1 - \sin\phi)}\, D \qquad (2.15)$$

where

ϕ is the internal friction angle,

σ_v is the vertical effective stress at depth x,

E_D is the dilatometer modulus, and

F_ϕ is the empirical stiffness factor (between 1 and 2).

Gabr and Borden (1988) proposed a model to evaluate the coefficient of lateral subgrade reaction K_{ho} from the DMT. They used the same continuous hyperbolic tangent function as O'Neill and Murchison (1983 equation 2.10) with the only difference being that K_{ho} is calculated from

$$K_{ho} = \frac{(P_o - \sigma_h)}{half\ blade\ thickness} \tag{2.16}$$

$$K_o = 0.376 + 0.095\ K_D - 0.00461\ (\frac{q_c}{\sigma_v}) \tag{2.17}$$

where

σ_h is the in situ lateral stress at rest,

P_o is the corrected dilatometer reading,

half blade thickness is 0.27 in (6.85 mm),

K_o is the lateral stress coefficient at rest,

K_D is the dilatometer lateral stress index,

q_c is the cone point resistance, and

σ_v is the effective vertical stress.

2.5.4 P-y Curves Based on PMT Methods

Robertson et al. (1985) suggested a method that uses the results of a pushed-in pressuremeter to evaluate p-y curves of a driven displacement pile. They multiplied the pressure component of the PMT curve by an α factor to obtain the correct p-y curve. This factor is applied to the pressure component of the PMT expansion curve. The critical depth is assumed to be four pile diameters as shown in Figure 2.8.

Robertson et al. reduced α near the surface assuming that the response is affected by the reduced vertical stress. To obtain the p-y curve, the pressuremeter curve is translated to the lift off pressure that is equivalent to the initial lateral stress around the pile. The stress is multiplied by the pile width and the strain component ($\Delta R/R$) is multiplied by the pile half width. For small strain conditions ($\Delta R/R$) is equal to ($\Delta V/2V$).

$$y = \frac{\Delta R}{R}\,\frac{D}{2} \tag{2.18}$$

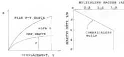


Figure 2.8 CPMT Multiplying Factor (Robertson et al.,1985).

Briaud et al (1984) proposed that the deflected shape of a laterally loaded pile depend on normal and shear stresses between the pile and the soil. Thus, the soil resistance is the sum of the front resistance load (Q) and the friction resistance load (F)

$$Q = (p - \sigma_h) D \alpha_n \qquad (2.19)$$

$$F = s_u D \alpha_s \qquad (2.20)$$

$$y = (\frac{a_i - a_o}{2 a_o}) D \qquad (2.21)$$

where

p is the applied pressure from PMT,

σ_h is the horizontal stress,

D is the pile diameter,

α_n is the normal shape factor (1 for square piles),

α_s is the shear shape factor (2 for square piles),

s_u is the average shear strength,

a_o and a_i are the initial pressuremeter radio and the radio during the test.

They defined a critical depth D_c, that is a zone below ground surface where the lateral soil resistance is reduced. The critical depth is a function of the pile rigidity, diameter, and limit pressure. The soil resistance from the pressuremeter curve is reduced by a β factor for pressures above 30 a_u (sands) and the front resistance is reduced by an α factor above the critical depth.

They developed a computer program PYPMT using pre-bored and self-bored pressuremeter results for bored piles. However, for driven piles they suggested using the reload curve. When using the CPMT, it is better to use the reload curve instead of the initial portion of the curve.

Schmertmann (1978) suggested that the CPT can be related to the PMT for lateral loading. He presented some correlations that relate q_c (cone resistance) to Young's modulus, E, and to the pressuremeter's limit pressure, P_L for sands and clays. Figure 2.9 shows an approximate p-y curve with two-line. A guide for estimating k_h and p_h is presented in Table 2.1 and equation 2.22 for flexible piles.

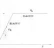


Figure 2.9 P-y Curve from CPT Correlation (Schmertmann, 1978)

$$(p_u)_{max} = 1.1\,P_L \qquad (2.22)$$

where

P_L is $q_c/10$ or $q_c/5$ for dense and loose sand,

E is $q_c/1$ or $q_c/1.5$ for dense and loose sand, and

D is pile diameter.

Table 2.1 Menard's Estimate of E/K_h in kg/cm^3 (Schmertmann, 1978)

Soil Type	Pile Diameter (cm)		
	30	60	90
Sand	9	18	22
Silt	13	26	37

The p-y curves derived from SPT N values utilize the correlations presented in Table 2.2 and Reese's ultimate soil resistance according equations. The angle of internal friction is the average from the surface to the depth of the p-y curve. These curves utilize K as the initial slope and a hyperbolic shape that converges with the ultimate soil resistance.

Table 2 2   SPT N Value Correlations for Cohesionless Soils

	Relative Density		
Parameter	Loose	Medium	Dense
SPT N	4-10	10-30	30-50
ϕ	28-35	35-42	38-46
K(pci)	20	60	125

2.5.5 T-z Curves Based on Soil Properties

FLPIER models the axial pile-soil interaction as a series of sixteen equally spaced nonlinear springs i.e. t-z curves The t-z curves are represented by a nonlinear hyperbolic equation presented by McVey et al (1989)

$$z = \frac{\tau_o r_o}{G_i} \left[\ln \left(\frac{r_m - \beta}{r_o - \beta} \right) + \frac{\beta (r_m - r_o)}{(r_m - \beta)(r_o - \beta)} \right] \qquad (2\ 23)$$

where

r_o is the radius of the pile (D/2),

r_m is the radius outward where the shear stress is negligible or zero, L is the pile length, μ is the Poisson's ratio of the soil usually 0 3

$r_m = 2.5\ \rho\ L\ (1-\mu)$ (Randolph and Wroth, 1978)

$$\rho = \frac{G_m}{G_i}$$

G_m is the soil shear modulus at the L/2,

G_t is the soil shear modulus at the pile tip,

$$\beta = \frac{r_o \tau_o R_f}{\tau_f}$$

τ_o is the shear stress at the pile-soil interface,

τ_f is the failure shear stress,

R_f is the hyperbolic constant usually 1.0, and

G_i is the initial soil shear modulus (Table 2.3)

The soil tip resistance is modeled by a single nonlinear spring at the base of the pile (McVay et al. 1989). The base of the pile is modelled as a rigid punch at a finite depth. The nonlinear spring at the pile tip is represented by equation 2.24.

$$w = \frac{P_b\,(1-\mu)}{4\, r_o\, G_i\, [\,1 - \frac{P_b R_f}{P_f}\,]^2} \qquad (2.24)$$

where

P_b is the mobilized tip load, and

P_f is the failure tip load.

Pile driving usually causes a densification of the sand close to the pile. In situ post-driving tests may be necessary to characterize soil properties and the state of stresses in the pile vicinity in order to evaluate load transfer mechanisms in sands. However, they are difficult to performed and their response must be related to pre-driving in situ tests. Averaged CPT friction resistance is used directly to determine the failure shear stress, τ_f in the vicinity of the pile. CPT cone resistance can be used to determine pile end bearing. The initial shear modulus is calculated using SPT N values as shown in Table 2.3 or indirectly by a Q_c/N correlation (Casper, 1995).

Pile failure shear stress and failure tip stress are based on SPT N correlations proposed by Schmertmann in FDOT Research bulletin 121. These correlations are presented in Table 2.4

Table 2.3 Initial Shear Modulus for Sand

Initial Shear Modulus G_i (psi)	Restrictions
$G_i = 500\ N$	$N \le 40$
$G_i = 20000$	$N > 40$

Table 2.4 Design Friction and End Bearing

Soil Type	Failure Shear Stress τ_f (tsf)	Failure Tip Stress σ_t (tsf)
Clean Sand	$\tau_f = 0.019$ N	$\sigma_t = 3.2$ N
Soft Limestone, very shelly sand	$\tau_f = 0.01$ N	$\sigma_t = 3.6$ N

CHAPTER 3
IN SITU SOIL TESTS AT PROJECT SITE

3.1 Introduction

Soil is a complex material with great variability due to a combination of geologic, environmental, and chemical processes making soil modeling a difficult task. This variability and in situ stress conditions have lead to the adoption of in situ tests as an appropriate method to determine soil properties for discrete-spring models.

Determination of soil properties can be based on in situ soil tests by a direct approach, indirect approach or a combination of both. The direct approach evaluates foundation performance mostly by empirical methods directly from in situ measurements without calculation of any intermediate soil parameters. The indirect approach leads to more rational design methods by correlating in situ results with common parameters such as friction angle, shear strength, unit weight, relative density, etc. These parameters are then applied to boundary solution problems (Campanella et al., 1989).

In laterally loaded piles, indirect approaches and 'default methods' such as those proposed by Reese et al. (1974) or O'Neill et al (1983) are often used together to determine p-y curves for sand. However, results obtained by using the 'default p-y curves' are highly dependent on the correct adoption of soil parameters and in most cases can predict excessive deflections and moments for the actual in situ soil conditions leading to higher costs in the design.

Four in situ test types were selected to determine soil properties at the project sites: standard penetration test (SPT), cone penetration test (CPT), dilatometer test (DMT), and cone pushed-in pressuremeter test (CPMT). The selection was based on availability, versatility, suitability for soil type, and literature recommendations for laterally loaded piles.

All of these tests have advantages and disadvantages with respect to one another. SPT is the most common in situ test that can approximate soil resistance with dynamic penetration and obtain a disturbed sample of the soil. Even though there are many sources of errors that limit its use, the SPT N value can be used as an index parameter to determine the approximate strength and compressibility of cohesionless soils for preliminary design purposes. SPT can be unconservative on soils containing coarse gravels, cobbles or boulders.

CPT provides a rapid method to determine the soil profile and estimate engineering properties with almost no influence of test personnel. The most useful parameters measured by the test are the tip resistance, q_c, and side resistance, f_s. CPT is more reliable than SPT and is becoming a more popular test in recent times. However, for cemented sands, SPT tends to break down cementitious bonds between soil particles during testing similar to pile driving while CPT produces less disturbance during testing resulting in abnormally high q_c values.

Pressuremeter tests can be used to determine in situ stress, deformability, and strength. PMT has a significant advantage for the design of deep foundations because of the relation of these foundation to the in situ horizontal stress (Kulhawy and Mayne, 1990). The cone pushed-in pressuremeter is a more recently type and can be operated more quickly without the need of a prebored hole.

Finally, dilatometer tests have been correlated to many soil properties and can provide a reasonable estimate of the horizontal stress and the overconsolidation ratio. DMT seems to be very useful within the elastic behavior range of soil. The blade-like shape reduces the shear and volumetric strain associated with other penetration tests. CPT, PMT, and DMT can be performed in a wide range of soils. However , they can not be used to penetrate very hard soils or gravel (SPT N > 40) or to obtain soil samples.

3.2 Preliminary In Situ Soil Tests

In addition to the SPT tests from Law Engineering (1989), the University of Florida has performed Cone Penetration (CPT), Dilatometer (DMT), and cone pushed-in Pressuremeter tests (PMT) at the project site. The cone pushed-in pressuremeter was modify by welding a 2 mm. thick ring between the cone and the pressuremeter section as a friction reducer to protect the Chinese lantern. The friction reducer ring creates a larger cavity that modifies the typical expansion curve for a pushed-in PMT. To compensate for this, the origin must be translated as in the pre-bored PMT or the resulting PMT curve is obtained after the reload. Thus, the initial modulus is similar to the slope of the reload curve. This reload modulus is very sensitive to the rate of loading and the number of readings taken during the unload-reload cycle. The PMT reload modulus is calculated by equation 3.1. Data were corrected by membrane stiffness, system compressibility, and hydraulic head.

$$E_R = 2\ (1 + \upsilon)\ \frac{\Delta P}{\Delta V}\ V_m \qquad (3.1)$$

Preliminary in situ tests were performed close to Pier 19 and the north abutment bent with the UF Cone truck. Prior to the test at Pier 16 N, the UF group proof-tested the Cambridge self-boring pressuremeter trailer and equipment at Pier 18 South, in particular a newly modified CPT depth counter. The trailer was used due to space constraints and the impossibility of using the cone truck on the pile cap. These tests showed that the CPT could not penetrate the stiff cemented sand layer at a depth of about 15 - 20 feet due to insufficient reaction for pushing and the limiting capacity of the trailer equipment. Post-driving CPT and CPMT results at Pier 18 S show a marked increase in cone bearing and friction resistance, and PMT's limit pressure in the sandy layer.

Figure 3.1 shows the anchor post-driving system to transfer a maximum of 25 kips uplift force to the pile cap through the rebar anchors (Vergara, 1982). This system was implemented and improved at Pier 18 South prior to the tests at Pier 16 North. Six in diameter PVC holes were placed through the concrete pier caps. Six ¾ in. anchor bars were left in the pile cap to transfer the load to the cap through an I beam. The plan to prevent the cone rods from buckling due to the unsupported length in the pile cap was (1) to place the rods and penetrate with the cone until the friction reducer is in the ground, (2) place four-in PVC pipes to the mudline, and (3) than backfill the PVC pipes with sand. This technique was successful.

Appendix A presents in situ test details at the project site. Figures A.1 and A.2 show a plan view and the profile of

SPT boring logs respectively performed by Law Engineering. Figures A.3 thru A.8 show the CPT profiles at Pier 19, North Abutment and Pier 18. Figure A.9 shows interpreted geotechnical parameters from DMT at Pier 19. Figure A.10 shows the calibration curves used during pressuremeter tests and Figures A.11 thru A.13 show pressuremeter corrected curves at Pier 19, North Abutment, and Pier 18 respectively. Tables A.1 thru A.7 show CPT and DMT soil interpretation for the profiles shown above.

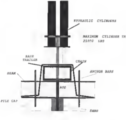


Figure 3.1 Anchor Post-driving System.

3.3 In Situ Tests at Pier 16 North and Vicinity

The process of pile installation in a group apparently increases the lateral stresses against the piles, thus increasing the side resistance capacity. Thus, pre-driving and post-driving in situ tests were performed to establish a relationship with pile performance at Pier 16 North.

According to the preliminary SPT tests at the vicinity of Pier 16 N before pile driving, the soil profile consists of the following:

A surface layer of slightly silty fine sand, with occasional shell fragments. The layer thickness ranges from 10 to 15 feet, SPT N values between 4 and 26 and ϕ between 30° and 38°.

Underlying the loose layer, a partially cemented fine sand with discontinuous layers of weakly cemented to cemented sandy limestone. SPT N values range between 12 and 100+ and ϕ between 40° and 46°.

A schematic view of the in situ test arrangement is shown in Figure 3.2. Cone PMT, CPT, and DMT, were performed at Pier 16 North before and after pile driving. The SPT N values were obtained from CPT correlations and by a FDOT crew. Figure 3.3 presents CPT and SPT profiles, Figure 3.4 presents the DMT corrected A and B readings, and Figure 3.5 presents the PMT modulus and the limit pressure profile for both conditions.

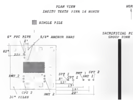


Figure 3.2 Plan View of In Situ Test Wells at Pier 16 North

3.3.1 Pre-driving In Situ Tests

The test site has about six feet of water and two well-defined layers of cohesionless soil. A loose layer of fine sand underlain by a layer of partially cemented sand that is difficult to penetrate with the truck or trailer.

A trestle was used for the pre-driving in situ tests at the test pile group site. Four five foot steel casings filled with sand were used to prevent the rods from bucking. The casings were filled after the rods were placed.

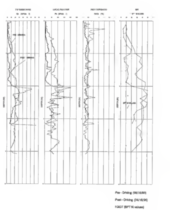


Figure 3.3 CPT and SPT N Values Before and After Pile Driving at Pier 16 North.

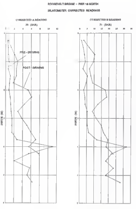


Figure 3.4 Dilatometer Corrected Readings Before and After Pile Driving at Pier 16 North

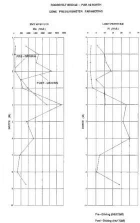


Figure 3.5 Cone PMT Modulus and Limit Pressure Before and After Pile Driving at Pier 16 North.

The CPT profile before pile driving is shown in Figure A.14 and its interpretation is presented in Table A.8. Figure A.15 shows the DMT profile for Test #1 and Table A.9 presents DMT interpreted parameters. Figure A.17 shows cone PMT corrected curves and Figure A.18 shows the thrust resistance profile during the CPMT penetration. Figure A.19 shows the soil profile and SPT boring log performed by FDOT. Table 3.1 presents a summary of pre-driving in situ tests parameters. Figure 3.6 shows the UF cone truck over the trestle at the pile test site before pile driving.

Figure 3.6 UF Cone Truck over Trestle at Test Site.

Table 3.1 Pre-driving Soil Parameters from In Situ Tests

Depth from Mudline	Cone Penetrometer Test			Dilatometer Test		Pressuremeter Test	
inches	Qc Mn/m²	Fs kn/m²	Fr %	Ed bar	Kd	E_M bar	P_L bar
20	1.89	2.50	0.13	63	10.1	--	--
40	1.67	4.94	0.29	87	5.4	61	1.5
60	4.79	17.36	0.36	109	5.2	65	1.6
80	5.54	16.73	0.30	96	4.0	84	1.6
100	3.26	10.94	0.33	108	4.4	466	2.1
120	5.37	24.65	0.46	133	4.6	595	2.8
140	4.90	15.20	0.31	206	5.4	--	--
160	20.54	61.28	0.30	536	19.8	3330	23.0
180	33.79	117.53	0.35	578	11.1	--	--
200	25.58	80.43	0.31	605	12.8	900	21.0
220	18.49	55.94	0.30	--	--	--	--
240	10.61	52.35	0.49	--	--	1370	25.0
280	--	--	--	--	--	409	7.0
320	--	--	--	--	--	241	5.5
345	--	--	--	--	--	841	19.0

3.3.2 Post-driving In Situ Tests

The Law Engineering report of Pile Group effect evaluation (1995) based on PDA and CAPWAP shows that the skin friction resistance increases a 100% due to densification of the sand around the pile by the driving of all the piles in a group.

A representative CPT post-driving profile is shown in Figure A.20 and its interpretation is presented in Table A.10. Table A.11 presents DMT interpreted geotechnical parameters and Figure A.16 shows the DMT profile within Pier 16 N. Figure A.21 shows PMT corrected curves for different depths. Finally, Table 3.2 presents a summary of geotechnical parameters obtained from in situ tests after pile driving.

Comparing pre and post driving values for the upper layer, the tip resistance increased by a factor of 3 to 4 and the friction resistance by 6 to 8. The friction ratio was between 1.2 and 2. The SPT N values correlated from CPT according to Robertson et al. increased 2.5 times. At this time, SPT tests were not performed after pile driving to correlate them to the pre-driving tests. The dilatometer corrected readings increased by a factor 2 to 3, the DMT modulus (E_D) by 2 to 3 and the horizontal stress index (K_D) by 4 to 5. The pressuremeter modulus (E_M) and the pressure limit (P_L) increased between 5 to 8 times. The cemented sand layer does not experience a marked increase, the maximum increments are less than 50 %.

Figure 3.7 shows a photograph of the UF trailer at Pier 16 North cap during an automated CPT sounding using a manual depth counter

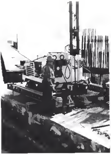


Figure 3.7 UF Trailer at Pier Cap

Table 3.2 Post-driving Soil Parameters from In Situ Tests

Depth from Mudline	Cone Penetrometer Test			Dilatometer Test		Pressuremeter Test	
inches	Qc Mn/m²	Fs kn/m²	Fr %	Ed bar	Kd	E_M bar	P_L bar
20	2.75	10.75	0.42	180	30	363	5.8
40	4.78	35.28	0.75	220	29	524	10.0
60	9.89	58.24	0.61	191	17	520	11.0
80	14.94	75.64	0.51	368	24	810	14.0
100	18.86	86.54	0.46	304	20	248	5.0
120	16.03	74.38	0.48	270	13	355	10.0
140	16.77	77.11	0.48	378	18	--	--
160	23.81	102.05	0.43	468	17	524	10.5
180	28.12	108.76	0.38	531	19	--	--
200	23.93	86.91	0.35	---	--	--	--
220	21.44	79.69	0.37	---	--	--	--
240	23.62	71.74	0.30	---	--	--	--
280	19.36	54.21	0.28	---	--	--	--
320	12.11	68.72	0.55	---	--	--	--

CHAPTER 4

LATERAL LOAD TEST PROGRAM

Presented in this chapter are the test layouts, procedures, instrumentation, and observations of the lateral load tests for the single pile and the test pile group.

4.1 Background

Originally, it was planned to lightly jack apart piers 14 North against 14 South. However, in order to obtain meaningful data these piers should be pushed close to failure, causing irreversible damage to the piles. Consequently, the test program was redesigned to fail a sacrificial free headed pile group using a fixed headed group for reaction, considering that a free headed pile group will deflect much more under a given load than a fixed head pile group. Finally, Pier 16 North was selected as the reaction pier assuming that sufficient reaction force would be available to fail the test group. Different scenarios were analyzed trying to predict the behavior and recommend a pile group configuration reaching an optimum condition, i.e. complete failure of the test pile group without damaging the reaction group. In the five scenarios considered Pier 16 North (fixed head) is the reaction:

Scenario A - 4 x 4 pinned 30" pile test group.

Scenario B - 3 x 5 pinned 30" pile test group.

Scenario C - 4 x 4 pinned 30" pile test group, similar to scenario A with reduced prestress force in piles.

Scenario D - 4 x 3 pinned 30" pile test group.

Scenario E - 4 x 4 pinned 24" pile test group.

FLPIER with its non-linear option for prestressed concrete piles was used to re-analyze the different scenarios. Soil properties that approximate lateral pile deflection in a preliminary report for a single 30 inch pile were considered (Hoyt, 1994). Figure 4.1 presents the load deformation response for the entire group and Figure 4.2 presents the load deformation response for a pile in the leading row. Finally, after reviewing the different scenarios with their advantages and disadvantages, FDOT (11/28/94) selected scenario A increasing the total load to 1000 kips or about 75 kips per pile in the leading row. This decision was based on pile dimension, group configuration, construction concerns, and trying to not alter the original project. Note that a load of 1000 kips would produce in the test group a deformation of about 4 times the deformation in the reaction pier.

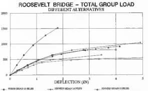


Figure 4.1 Load-Deflection Curves for the Pile Test Group.

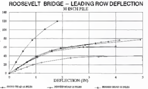


Figure 4 2 Load - Deflection Curve for the Leading Row

4.2 Lateral Load Test Program

The program consisted of two fully instrumented lateral load tests: a single isolated 30 inch pile and a sacrificial sixteen pile group. Both tests were performed using Pier 16 North as a reaction. Instrumentation consisted of a total of 17 steel pipes (40 ft length) and a total of 648 strain gages. A schematic plan view of both tests is provided in Figure 4.3. Information such as lateral and vertical deformations, bending and axial strains, forces per pile, vertical and horizontal pile cap rotations, etc, are presented in Appendix B, C, and D.

4.2.1 Single Pile Test Layout

The single isolated pile load test consisted of a fully-instrumented 30 inch prestressed concrete pile that was loaded until failure i.e. a rapid progressive lateral movement of the pile under constant or decreasing load. This prestressed concrete pile was voided with a 18" PVC pipe. A 14" steel pipe (3/8" thick) and a slope inclinometer casing were inserted into the void and grouted into the pile after driving. Spacers were attached to the inserted pipe and the casing to ensure proper centering in the pile. The instrumented pipe had 9 levels of strain gages (only for bending) attached to the

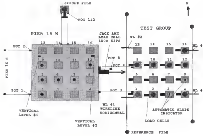


Figure 4.3 Plan View of Pier 16 North and Test Pile Program.

outside surface and placed every 36 inches below the mudline except the last one at 72 inches. Every level had four gages i.e. full bridge configuration with cables connected to a terminal strip at each level. Lead wires ran through the 14 inch pipe to the data acquisition system. Deflections and slopes at the top of the pile were measured with the slope inclinometer and two potentiometers. Figure 4.4 presents a schematic layout of the single pile load test with the soil properties estimated from in situ soil tests at pier vicinity.

4.2.2 Pile Group Test Layout

The load test pile group consisted of sixteen 30 inch sacrificial piles that were subjected to 1080 kips using Pier 16 North as a reaction. The sacrificial pile group had 10 fully instrumented piles with the same instrumentation as the single pile (pinned head), while the reaction group had 6 piles fully instrumented (inclinometer and 7 levels of strain gages). In addition to the original strain gages for bending, each level of the reaction group piles had pairs of quarter bridge strain gages that were added to measured the axial deformations. Figure 4.5 presents a schematic view of the pile groups and the soil characteristics estimated from in situ tests performed between pier 16 north piles.

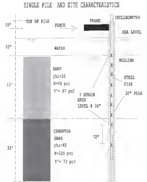


Figure 4 4 Profile of the Single Pile and Soil Characteristics

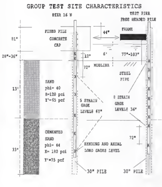


Figure 4.5 Profile of the Pile Groups and Site Characteristics.

The reaction pier is connected though an 81 inch thick concrete cap that "fixed" the piles heads against rotation while the sacrificial or pinned pile group was connected by a steel frame. One of the difficulties is to establish pile-head conditions for pier 16 N in the field to stimulate those of the foundation that is being designed; ground clearance was considered to eliminate any soil-cap interaction effects and to facilitate load-transfer interpretation; all loads applied to the cap are resisted by the piles only (no friction), there was about 2.5 feet between the mudline and the pile cap.

Deformations and rotations at the top of the piles were measured using slope inclinometer readings and potentiometers. Applied total loads were measured with a 500 ton load cell and individual loads were measured with thirteen 50 ton load cells on every instrumented pile. Vertical and horizontal movements were measured at the pile cap corners assuming Pier 16 South as a reference i.e. did not move during the test. Bending and axial strains were recorded in six of the sixteen piles in pier 16 north, only bending strains were recorded in ten of sixteen piles in the sacrificial pile group. These readings are presented in Appendices C and D. All this instrumentation is shown in Figure 4.3

4.2.3 Pile Cross Section

The pile cross section and some material specifications for instrumented piles in the sacrificial group and reaction group are presented in Figure 4.6. The pile rigidity (E I) for the pinned head section was 340 10^6 kip-inch^2 and 335 10^6 kip-inch^2 for the fixed head section. The cracking moment was considered 7000 kip-in and the ultimate moment 12500 kip-in.

Concrete (Cement Class V): $f_c = 6000$ psi (design at 28 days). However, the piles were submerged for more than 10 months before the test. According to Nilson (1987), $f_{c\,365} = 1.15\, f_c$

$f_{c\,365} = 7200$ psi and $E_c = 4836$ ksi

Prestressing steel: ASTM A-416 Grade 270 Low relaxation with a prestressing stress after all losses of 193.4 ksi.

24 # ½" ϕ (strand area 0.167 in^2) at 32 300 lbs. each

$E_s = 27600$ ksi

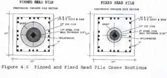


Figure 4.6 Pinned and Fixed Head Pile Cross Sections

4.3 Instrumentation

The behavior of the surrounding soil and the pile itself could be discerned by analyzing the instrumentation data through monitoring loads, displacements, strains and shapes with the instruments described in this section.

4.3.1 Strain Gages

The strain gages were installed on the exterior of the pipes by Measurements Group, Raleigh, N.C. The gages were calibrated prior to the installation considering the flexural rigidity of the steel pipe cross section. The gages used for bending strains were MM CEA-06-500UW-350 gages (full bridge) and for axial load were MM CEA-06-500UW-350 gages (quarter bridge). The strain gages were bonded to the pipe surface with M-BOND 200 adhesive and protected with M-COAT D. Originally, CEA-06-500UW-120 gages were specified in a full bridge configuration i.e. four gages per level. However, it was decided that 350 ohm gages could give more accuracy due to the higher excitation voltage. Since, there were 11 instrumented pinned piles (10+1) in both tests and 6 instrumented piles in the reaction group with 9 and 7 levels each, respectively, there are 141 levels of bending strain gages and 42 levels of axial strain gages. Each bending gage level consisted of a pair of gages on each side of the pipe (full bridge configuration). The axial gage levels consisted of pairs of quarter bridge strain gages to measure tension and compression. All the gages were sensitive to strain in the vertical direction. The bending strain gages were installed in a way to cancel axial and temperature effects. They were positioned at the extreme fiber axis i.e. 90° from the neutral axis or parallel to the loading direction as shown in Figure 4.7 The axial strain gages were positioned at the neutral axis to cancel bending and temperature effects at the same elevations as the bending gages. Figure 4.8 shows a photograph of the strain gages installed on the pipes used for bending and axial strains. The strain was computed as

$$\varepsilon = \frac{4\ Measured\ \ Voltage}{G.F.\ Applied\ \ Voltage} \tag{4.1}$$

where

GF is the gage factor (2.09),

V_o is the applied voltage (10 volts), and

ΔV is the measured voltage due to strain.

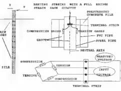


Figure 4.7 Strain Gages - Full Bridge.

Figure 4.8 Photograph of Axial and Bending Strain Gages.

4.3.2 Slope Inclinometers

The main purpose of an inclinometer is to monitor the changes in inclination of the casing from its original position. All instrumented piles of the load test program had a slope inclinometer casing for measuring the deflected shape at every incremental load. It was suggested to fill the slope inclinometer casing with water prior to grouting to lessen heat of hydration effects. One axis of the SI casing was aligned with the direction of the load.

This project include two types of inclinometers, manual and automatic. Twelve readings of a meter depth were taken using the manual slope indicator and only six readings using the automatic slope indicator. Data collection was made with a multi-channel/multi sensor for the automatic inclinometers and manually for the others. Reduced data are presented in Appendices B, C and D for both tests.

4.3.3 Load Cells

Ten load cells of at least 100 kips (50 tons) were utilized for each instrumented pile in the test group, in addition to the 1000 kip load cell (500 tons) on the jack. Three additional load cells were installed in Pile 5, 6, and 7 in order to obtain more information.

4.3 4. Data Acquisition System

The electronic data acquisition system, System 4000, was provided by Measurements Group, Inc Its function was to store strain gage readings from the gages at each level for every incremental load. During the pile group test, there were 132 levels for bending strain gages, 42 levels for axial strain gages with 2 channels per level, 13 load cell channels, and 3 potentiometer channels, which result in 232 channels or measurements per reading Finally, there were 1751 readings taken during the group test at 15 second increments.

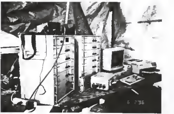


Figure 4.9 Photograph of the Data Acquisition System.

4.4 Description of Lateral Load Tests

The lateral load test program was performed by Law Engineering under the supervision of the University of Florida. Presented in this section are the procedures and observations made during the load tests.

4.4.1 Load Test of Single Pile

The load test of the single pile began at 1:15 p.m., November 7, 1995. An initial load was applied to the pile and maintained for 15 minutes. Three readings were taken for each incremental load. The load increments are presented in Table B.1. The load test concluded at 6:05 p.m. The failure load corresponded to 72 kips. A photograph of the single pile during testing is shown in Figure 4.10.

During the course of the test the following problems occurred.

a) Only 3 of the 9 strain gages level functioned properly. It is believed that due to the lack of proper care of the wires, coupled with severe weather caused the wires and the terminal strips to abrade against the piles and be submerged. Consequently, water enter the nicks in the insulation either shorting or altering the resistances. Although the strain gages were read in the field prior to the placement in the pile, they were not reread following completion of grouting. Also a significant delay during grouting occurred.

b) Anticipated failure load/deformation indicated that a 100 kip jack with a 6-inch ram stroke would be required for the test. Unfortunately the jack only had a stroke of 2.5 inches, which required successive shimming and blocking to deform the pile. Also, the load cell quit functioning at 60 kips. The jack and the load cell were calibrated after the testing. The measured load at each load incremented was obtained from the calibration submitted by the contractor and presented in Figure B.1.

c) Finally, the data acquisition system did not work due to hard drive failure and the data were recorded manually.

Despite these difficulties some important information was obtained and is analyzed in Chapter 6. It is intended to schedule a single pile load test, by loading test Pile #3 in the opposite direction of the group test (West). A single pile layout is presented in Figure B.7. All records of the single pile load test are presented in Appendix B.

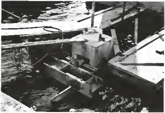


Figure 4.10 Photograph of the Single Pile during Testing

4.4.2 Load Test of Pile Group

After the difficulties experienced during the single pile test, special precautions were taken during the installation of the instrumented pipes. The strain gages and the gage leads were re-coated to protect them from moisture and the resistances (350 ohms or 262 ohms) were checked prior to and after grouting. Also, the strain gage connections were protected by capping the wire ends and wrapping with duct tape. To prevent shimming, the extension (6 inches) and the capacity (1000 kips) of the hydraulic jack were checked

Spherical bearing connections for individual load cells and for the main 500 ton load cells were installed to allow pile movement without introducing eccentricity. Working platforms were constructed over a trestle, independent of the loading frame, to allow the personnel to read slope inclinometers and wirelines without any risk. Pile 15 was grouted and reinforced due to some damage produced during driving. Finally, prior to the load test a 125 kip load was applied to the piles to check the equipment and data acquisition system.

The group load test was performed under the direction of Dr. Brown and Dr. Townsend. It began at 11:10 a.m., June 13, 1996, and concluded at 4:45 p.m. The initial load of about 125 kips was applied to the pile group and maintained for 30 minutes. It was intended to maintain a constant load at each load level. Three readings were taken: load A at 1 min., load B at 10 min., and load C at 30 min. after the loading. Only one reading per load, at load B or 10 min., was made with the manual slope indicators. Nine load increments of about 125 kips and four unload increments of about 300 kips were taken given a total of thirteen loads. The maximum load corresponded to 1200 kips. At load #4 the load was decreased to zero and reloaded to the next load increment. The loading sequence is presented in Table C.1.

There were 13 individual load cells, i.e. one per each instrumented pile and three additional load cells, and one group load cell. These load cells were read automatically by the data acquisition system. Four wirelines between H-pile reference piles with mirror-mounted scales with 0.01" divisions were placed at the four corners of the test pile group. Two vertical levels were taken from Pier 16 South to observe the vertical displacement of the pier. Five horizontal displacement transducers (potentiometers) were mounted to monitor the inter-pile displacement and the pier movement. Two in-place inclinometers were used on pile 3 and 8 that recorded automatically at 1, 10, and 30 minute reading intervals. They only recorded the first six meters from the top of the pile. Fourteen conventional inclinometers were used to record the deflection and rotation of the first 12 meters after the 10 minute reading of each load, and after load 11 B and 13 B. The instrumentation details are shown in Figure 4.3. Finally, a video was taken with relevant aspects of the test for documentation purposes.

A photograph of the sixteen pile group after the installation without the loading frame is shown in Figure 4.11. A schematic view of the loading frame and with the load cells is shown in Figure 4.12. The test pile group and Pier 16 N during testing are shown in Figures 4.13 and 4.14, respectively. Purged information is presented in Appendices C and D for the test pile group and the reaction pier, respectively.

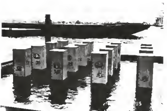


Figure 4.11 Photograph of the Pile Group After Installation.

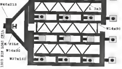


Figure 4.12 Plan View of the Loading Frame and Load Cells.

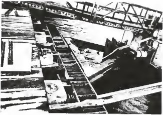


Figure 4 13 Photograph of the Pile Group Test During Testing

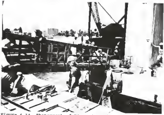


Figure 4 14 Photograph of Pier 16 North During Testing

CHAPTER 5

EVALUATION OF LOAD TRANSFER CURVES

Presented in this chapter are the load transfer curves derived from instrumentation readings and in situ tests. The 'measured' transfer curves from the instrumentation are compared with the predicted curves obtained from in situ tests at similar depths. Results from both the single pile load test and the pile group load test are included.

5.1 Lateral Load Transfer Curves

As mentioned previously, lateral soil response is represented by an elastic foundation with nonlinear springs to transfer the horizontal load from the pile to the soil. These nonlinear springs are often called p-y curves. It is assumed that the soil resistance parallel to the axis of the pile is negligible and the behavior of the soil at a particular depth is 'independent' of the soil behavior at all other depths. In this section, it is intended to find the correct p-y curves that best simulate the load-deflection curve at the top of the pile, considering that the bending moments do not vary significantly with different methods. The comparison is based on the slope of the initial portion of the curve, the ultimate soil resistance and the curve's shape.

5.1.1 P-y Curves Obtained from Instrumentation

P-y curves presented herein are derived from the lateral load tests by instrumentation, i.e., bending strain gage measurements and inclinometer deflections. P-y curves based on strain gages are mentioned as the 'measured or true' p-y data according to previous experience.

a) P-y Curves Obtained from Bending Strain Gages

To obtain moments from the strain gage readings, the strain must be related to the curvature and the curvature to the bending moments. The curvature is the measured strain divided by the distance between the gages (pipe diameter) and the gage factor. Before cracking the pile rigidity is constant and the curvature is proportional to moments. But after cracking, the nonlinear behavior of the pile must be considered. Nonlinear FLPIER can be used to calculate the curvature of a concrete pile section, either reinforced or prestressed pile, by fixing the pile at the bottom and applying moments at the top.

The bending moments were fitted to polynomial equations using the least-square polynomial regression method that was implemented in a spreadsheet Third order polynomial equations with a maximum of 7 points for each curve were used to evaluate the p-y curves It is supposed that a low order polynomial can model better the soil reaction To obtain a p-y curve at a specific depth, the polynomial is fitted with the same number of points above or below. Deflection y(x) was obtained by double integration and the soil reaction p(x) was determined by double differentiation of the moment curves. Inclinometer measurements were used as a complement to obtain integration constants, i e pile head rotation and deflection.

Figure 5.1 shows the 'measured' p-y curves for the single pile obtained using bending strain gages for the previously specified depths Unfortunately for the single pile only three of the nine strain gage full bridges worked However, the results are reasonable and are of the correct order of magnitude Polynomial regression seems to work only during the elastic range of the pile i e when the pile did not crack and its rigidity is constant After cracking it is assumed that the soil resistance follows the previous tendency, i e a perfect plastic curve for the upper curves or a linear increase for the bottom curves The initial portion of the p-y curves for the sand layer varies from 1200-5000 psi and the ultimate soil resistance from 300-1500 lbs/in approximately

b) P-y curves from Inclinometer Readings

Slope indicator deflection data from the lateral load test provide the deflection from the top of the pile to 11 meters depth. The finite difference method for case 1 (P_t and M_t known) presented in COM624P was implemented in a spreadsheet, in which the soil modulus was varied at different depths until the actual pile deflection matched the predicted pile deflection. P-y curves were formed with these soil moduli at different load increments. After cracking, the pile rigidity EI was decreased according to the resulting bending moment presented in Figure 8.5. This method presents very accurate p-y curves for the upper layer until 120 inches depth. The initial portion of the p-y curves for the sand layer varies from about 3000-8000 psi and the ultimate soil resistance from about 200-1500 lbs/in resulting in a very stiff initial slope with a hyperbolic shape. For deeper p-y curves the accuracy decreases in that the inclinometer readings are very small and the inclinometer only reaches 11 meters depth. However, the load-deflection curve obtained using these p-y curves matches the measured load-deflection as shown later in Figure 6.4. This method seems to be more accurate than the one that utilizes inclinometer readings with the non-dimensional coefficients for cohesionless soils, because the soil modulus for any depth is independent of any depth function. However, this method only gives an order of magnitude of the soil resistance with deformation. They are not exact especially after cracking and at deeper layers.

Figure 5.1 shows p-y curves based on instrumentation readings for a 'representative' depth (60" depth) for both strain gage and inclinometers readings. Figures 5.2 through 5.5 show the 'measured' p-y curves obtained for a representative pile of each row of the test pile group at 60" depth using instrumentation readings. Figure 7.1 defined the row position and pile numbering of the test pile group and the reaction pile group. Figures E.1 through E.5 show p-y curves based on strain gage readings for 20, 40, 60, 80, 120, 160, and 200 inch depths. Figure E.6 shows these p-y curves obtained using inclinometer readings for the single pile. Figures E.7 and E.8 show P-y curves for a pile in the leading row (pile 16) and a pile in the trailing row (pile 9) of the test pile group, respectively.

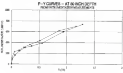


Figure 5.1 P-y Curves Based on Instrumentation Measurements for the Single Pile

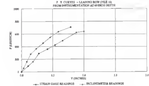


Figure 5.2 P-y Curves Based on Instrumentation Measurements for a Leading Row Pile (Test Pile Group).

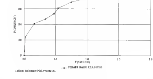


Figure 5.3 P-y Curves Based on Instrumentation Measurements for a Middle Leading Row Pile (Test Pile Group).

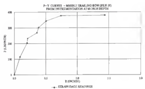


Figure 5.4 P-y Curves Based on Instrumentation Measurements for a Middle Trailing Row Pile (Test Pile Group)

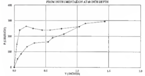


Figure 5.5 P-y Curves Based on Instrumentation Measurements for a Trailing Row Pile (Test Pile Group)

5.1.3 P-y Curves Based on In Situ Tests

In situ tests were used by direct or indirect form for the determination of p-y curves. The Reese, Cox, and Koop method and O'Neill and Murchison method are considered as 'default' methods in current programs. Soil properties such as ϕ, γ', and K were estimated indirectly through SPT and CPT tests. For pre-driving soil conditions, the angle of internal friction of the upper layer was 32°, the effective unit weight was 57 pcf, and the modulus of subgrade reaction was 60 pci as recommended for medium dense sands below the water table in COM624P. For the bottom layer, ϕ was 42°, γ' was 73 pcf, and K was 125 pci as recommended for submerged dense sands. For post-driving soil conditions, the angle of internal friction of the upper layer was increased to 40°, the effective unit weight to 65 pcf, and the modulus of subgrade reaction to 120 pci. For the bottom layer, ϕ was 44°, γ' was 75 pcf, and K was 180 pci. Figure 5.10 and 5.11 compare p-y curves obtained from different in situ tests at a particular depth (2 D) before and after pile driving, respectively.

The characteristic shapes of the p-y curves computed with the Reese and O'Neill methods are very similar. Reese presents three portions with a very stiff initial part and O'Neill presents a hyperbolic shape. The major difference is the computation of the ultimate soil resistance. Reese's method seems to be more accurate in the determination of the ultimate soil resistance for the same ϕ and γ'. However, the recommended values for the modulus of subgrade reaction seem to be very conservative. Figures E.9 and E.10 present p-y curves using the Reese method for pre-driving and post driving soil conditions, respectively. Figures E.11 and E.12 present the O'Neill method for both conditions.

The initial slope of the p-y curves of both dilatometer methods are very reasonable. However, the ultimate soil resistance directly depends on the angle of internal friction and can be very unconservative, especially if the ϕ angle is derived directly from the dilatometer. Gabr recommends ϕ angles derived from CPT correlations. However, they also seem to be very high. Gabr's method employs Reese's equation for the ultimate soil resistance with a hyperbolic shape while Robertson employs O'Neill's equation. Both methods were developed for the linear behavior of a pile. Figures E.13 and E.14 present p-y curves using the Robertson et al.(1989) method for pre and post driving conditions. Figures E.15 and E.16 present p-y curves using the Gabr and Borden (1988) method.

P-y curves developed from PMT results have the advantage of an accurate determination of the ultimate soil resistance especially for the Robertson et al. method. Briaud's ultimate soil resistance seems to be very high, probably because this method was developed for a pre-bored pressuremeter test. Both methods are inappropriate for the calculation of the initial slope of the curve, due to the protector ring added to the tip. The reload modulus can be more appropriate with some precaution, because it is very sensible to rate of unload /reload and the PMT correction curves. According to inclinometer p-y curves the initial slope must be 2 to 4 times the slope of the reload curve. The 'protector ring' at the PMT cone may be the cause of this problem. Figures E.17 and E.18 present p-y curves for pre and post driving conditions according to the procedure of Robertson et al. (1985) that uses the results of a pushed-in pressuremeter for driven displacement pile. P-y curves obtained from PYPMT program developed by Briaud et al. (1984) using PMT results are presented in Figures E.19 and E.20.

A set of p-y curves generated using rough correlations between CPT and PMT suggested by Schmertmann (1978) are presented in Figures E.21 and E.22. Considering that for medium dense to dense sands, q_c/P_L is estimated as 12, and E/K_o as 20 cm (Table 2.3), this method seems to be a good initial estimate but is not as precise as it should be. Figure E.23 shows the p-y curves obtained using the SPT N values from the vicinity of Pier 16 N i.e. pre-driving condition and Figure E.24 shows the p-y curves obtained SPT N values correlated from CPT post-driving results. The method correlates N with the angle of internal friction and the coefficient of subgrade reaction as shown in Table 2.4. The ultimate soil resistance is calculated using Reese's equations and the shape is assumed to be hyperbolic.

In summary, for the surface layer (from 0-160 in.), the measured curves show a very stiff initial slope i.e. between 2500 to 8000 lbs/in/in. and a ultimate soil resistance between 300 and 1200 lbs/in. If it is compared with the in situ p-y curves shown in Appendix E, the initial slope can be related to the dilatometer modulus (E_d) and the ultimate soil resistance directly related to the PMT limit pressure. Figures E.25 and E.26 present a DMT/PMT method that utilizes the 'measured' p-y curves as references and all the concepts presented before. The shape of the curve is linear from zero to 1.1 mm. (DMT B reading) and hyperbolic from 1.1 mm thereafter. The initial slope is related to the dilatometer modulus multiplied by a modulus factor F_e (between 2 and 3) and the PMT limit pressure is multiplied by an α factor according to Robertson et al. (see Figure 2.5). The method relates the dilatometer modulus to the elastic region and the PMT limit pressure to the plastic region of the load-deflection curve without the assumption of any soil parameters. This method has the advantage that no soil parameter has to be assumed, but has the disadvantage that two time consuming in situ soil tests are needed.

With respect to the soil densification due to closely-spaced pile driving, assuming that the shape remains the same, for the surface layer (from 0-160 in.), the in situ curves show a very stiff initial slope i.e. between 6000 to 10000 lbs/in/in and an ultimate soil resistance between 500 and 6000 lbs/in. The initial slope of the curve for the surface layer is 2 to 3 times as much as in the pre-driving p-y curves according to the dilatometer modulus presented in Table 3.2 and 3.4. The ultimate soil resistance increased 2 to 4 times the resistance of the pre-driving according to the PMT pressure limit.

Analyzing the p-y curves obtained from instrumentation, the lateral behavior of the pile group seems to be better represented by the pre-driving in situ soil conditions, the post-driving p-y curves were found to be stiff. Furthermore, these values are affected by the position of the pile inside the group. For the surface layer (from 0-120 in.), the measured curves from a leading pile show an initial slope between 600 to 3000 lbs/in/in and an ultimate soil resistance between 300 and 1000 lbs/in. The middle leading row presents an initial slope between 1000 to 4000 lbs/in/in and an ultimate soil resistance between 300 to 800 lbs/in. The middle trailing row presents an initial slope between 600 to 2500 lbs/in/in and an ultimate soil resistance between 350 to 500 lbs/in. Finally, the trailing row curves present an initial slope between 600 to 2000 lbs/in/in and an ultimate soil resistance between 100 to 300 lbs/in

Comparing with respect to the pre-driving soil values the initial slope decreases in all the rows between 2 to 3 times without a great difference between each other. However, the ultimate soil resistance decreases consistently by rows between 1.0/0.83 in the leading row, 1.0/0.4 in the middle rows, and 0.4/0.25 in the trailing row. Even though, these 'measured' curves are only an approximation this can give an idea of the group effects. It seems that the p-y multipliers are determined by the reduction of the ultimate soil resistance in the upper layer.

The values utilized in the analysis are presented in Tables 3.1 and 3.2 to summarize pre-driving and post-driving soil conditions respectively. More details of the in situ soil information are presented in Appendix A and the in situ p-y curves using both soil conditions are presented in Appendix E.

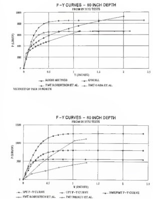


Figure 5.6 P-y Curves from In Situ Tests Before Pile Driving at 60" Depth

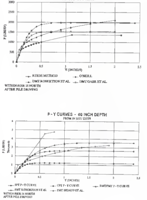


Figure 5 7 P-y Curves from In Situ Tests After Pile Driving at 60" Depth

5 2 Axial Load Transfer Curves

Axial soil response also can be represented by an elastic foundation with nonlinear springs to transfer the axial load from the pile to the soil. These nonlinear springs are often called t-z curves The values obtained from axial strain gages in the reaction pile group are compared with the predicted based on in situ tests

5.2.1 T-z Curves Obtained from Axial Strain Gages

T-z curves were obtained only in the reaction pile group of Pier 16 North, where six instrumented piles had axial strain gages It is not possible to calculate the maximum skin friction (τ_{max}) due to the small deformation. However, the initial shear modulus G_i is calculated with the axial strains obtained from Pile 3 and Pile 12 for load 3 C These values ranged between 3500 to 13500 psi for the first 120 inches. They are in between the recommended values using pre and post driving soil conditions

5.2 2 T-z Curves Based on Soil Properties

Figure 5 8 show t-z curves based on equation 2.23 and pre-driving SPT N values. Post-driving SPT N values have not been performed. However, they can be obtained from CPT correlations for the upper layer. CPT sleeve friction values can be used directly to determine the failure shear stress, τ_f, by averaging all CPT friction values in the vicinity of the pile node (Casper, 1995). There are two well defined layers, a medium dense sand layer with an average N value of 8 before pile driving and a dense cemented sand layer with and average N value of 20. To account for the densification during pile driving, a factor of 2.5 is introduced to the sand layer and about 1.0-1.5 for the cemented sand. The initial shear modulus presented in Table 2.3 is determined from SPT N values correlated from CPT readings according to Robertson correlation. Figure 5.9 shows the same t-z curves but calculated with post driving CPT values. For post-pile driving soil conditions, a q_c of 4 MN/m^2 and q_c of 20 MN/m^2 have been considered for the upper and bottom layer, respectively. Poisson's ratio is taken as 0.3 and the hyperbolic constant as 1.0.

Finally, the failure shear stress and the ultimate tip stress were calculated from SPT N values at the tip vicinity, that is about 30 (Table 2.4). Figure 5.10 shows the pile tip load-deformation curve with a failure tip load of 700 kips (Law Engineering, 1995).

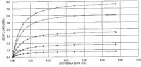


Figure 5.8 T-z Curves Based on Pre-Driving SPT N Values.

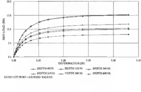
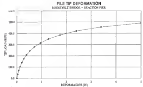


Figure 5.9 T-z Curves Based on Post-Driving CPT Values.



SPT N VALUES BEFORE PILE DRIVING

Figure 5 10 Pile Tip Load-Deflection Curve Assumed for the Analysis

CHAPTER 6

PREDICTION AND PERFORMANCE OF THE SINGLE PILE SUBJECTED TO LATERAL LOAD

Presented in this chapter is a summary of the results of the static single pile lateral load test as well as a comparison with the behavior predicted by the computer programs referred to in Chapter 2 utilizing different methods to compute p-y curves. The chapter also includes load-deflection curves, load-bending moment curves, and load-maximum moment curves. The in situ tests and the lateral load test results are presented in Appendix A and B, respectively.

6.1 Prediction of Load-Deflection Response

As mentioned previously, the lateral load was applied in 8 load stages to 72 kips, and unloaded in 4 stages as presented in Table B.1. The measured pile head loads with their corresponding maximum deflection (at 15 minutes) are shown in Figure 6.1. The difference between the curves is due the fact that they correspond to different levels i.e. LVDT level and Inclinometer level. The load was maintained constant

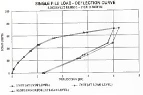


Figure 6.1 Measured Load-Deflection Curve for the Single Pile

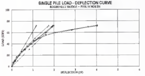


Figure 6 2 Predicted Load-deflection Curve Using Different Programs and 'Measured' P-y Curves while the deflection was recorded. As can be seen, after cracking i.e. bending moment about 7500 kip-in and lateral load about 50 kips, the rate of deflection increased until 'yielding' (12000 kip-in). A layout of the single pile test with all the instrumentation is shown in Figure B.7.

This variation is also compared in Figure 6.2 using Poulos's elastic solution (Poulos, 1971) and the programs FLPIER, COM624P, PIGR3D and GROUP with the 'measured' p-y curves obtained from inclinometer readings. The FLPIER program is represented by two options that consider the prestressed concrete pile as a linear elastic material and as a non-linear material. Figure 6.3 compares the measured load-deflection curve with that predicted from FLPIER using p-y curves obtained from instrumentation, i.e. strain gages and inclinometer readings. Figure 6.4 shows the deflection along the pile length measured from the load level i.e. 82 inches above the mudline for two loads, 30 and 60 kips.

Finally, Figure 6.5 shows the load-deflection curve predicted using FLPIER non-linear option with the p-y curves obtained from the in situ tests for pre-driving conditions. These p-y curves are presented in Appendix E.

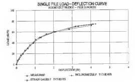


Figure 6.3 Load-deflection Curves Using FLPIER and P-y Curves from Instrumentation.

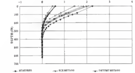


Figure 6.4 Deflection Along the Single Pile at Certain Loads

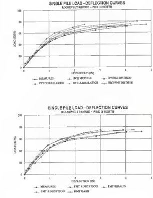


Figure 6 5 Load-deflection Curves Using FLPIER and Pre-driving P-y Curves

6.2 Prediction of Bending Moments

The relationship between the maximum bending moment and the pile-head load for Poulos' solution and the programs mentioned before is presented in Figure 6.6. This relationship is also compared in Figure 6.7 with the measured maximum bending moment and the predicted moments using FLPIER and the p-y curves derived from the instrumentation. The 'measured' maximum moments were calculated using a third degree order polynomial and the strains recorded. Figure 6.8 shows the variation of normalized bending moment i.e. moment/pile load along the pile depth in order to compare these curves for different load increments using inclinometer p-y curves.

Maximum bending moments obtained utilizing FLPIER for nonlinear material and the nine in situ methods of p-y curves are plotted for different pile load increments in Figure 6.9. The variation of bending moment along the pile length is shown in Figure 6.10 for a load of 30 kips. This figure shows not only the maximum bending moment at 30 kips but also its depth from the mudline and its shape.

The measured moments were obtained from the strain gage readings presented in Table B.3. Figure B.3 shows the moment thrust interaction diagram and the moment curvature curve, while Figure B.5 shows the moment pile rigidity curve for the prestressed pile section utilized.

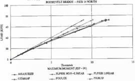


Figure 6.6 Maximum Bending Moment versus Load Using Different Programs.

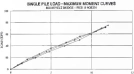


Figure 6.7 Maximum Bending Moment versus Load Using P-y Curves from Instrumentation.

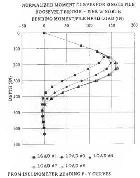


Figure 6.8 Normalized Moments Along the Pile at Certain Loads

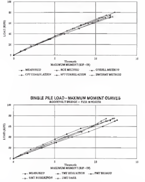


Figure 6.9 Pile Head Load versus Maximum Bending Moment Using FLPIER and Pre-driving P-y Curves

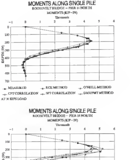


Figure 6.10 Bending Moments Along the Pile Using FLPIER and Pre-driving P-y Curves

6.3 Comments on the Single Pile Performance

The pile begins to deform plastically after 50 kips, resulting in an increasing deflection rate at similar load increments. The pile cracking moment seems to be an important value and depends only on the pile section and the axial load. For the pile tested it is about 7500 kip-in. The maximum load of the lateral load test was about 72 kips. This is not exact due to the problems described previously. The pile had an unrecovered deformation of 1.2 inches at the load level.

All these programs model the soil as a series of closely-spaced discrete springs with no coupling between each other i.e. unit load transfer model or Winkler approach. For a single pile with the same p-y curves (inclinometer p-y curves) and without pile-soil-pile interaction, the prediction is very similar for linear elastic pile properties. FLPIER for nonlinear predicts very accurately after cracking. Soil modulus values of 0.5 ksi and 1.0 ksi were used for Poulos's elastic solution. These values are recommended for medium and dense cohesionless soils. The solution is valid when the pile and the soil are in the elastic range, less than 20 kips. It is not accurate to assume that the soil modulus remains constant, it decreases with the deflection.

The inclinometer p-y curves provide a very accurate load-deflection curve much better than the strain gage p-y curves, in this case as shown in Figure 6.3. However, both give very similar load-deflection curves.

The deflection along the pile is determined by deflection at the top with very similar shape as shown in Figure 6.4. Since the load-deflection curve is more sensitive to a prediction method than the variation of bending moments, it is important to analyze the in situ p-y methods to establish a criteria. Shallow p-y curves until 120 inches depth (4D) have the major influence in the load-deflection curve in this case. The soil modulus that is proportional to the coefficient of subgrade reaction determines the initial part of the load-deflection curve. The ultimate soil resistance of the upper layer influences load-deflection at larger displacements with a transition between both conditions. Dilatometer methods provide a good approximation in the elastic region and Robertson's PMT method in the plastic region. The DMT/PMT method tries to join both methods with good results. Reese's procedure provides a very good approximation if the correct parameters were taken, for this case a high modulus of subgrade reaction and relatively low angle of internal friction. The SPT p-y curve method also provides a good approximation due to the fact that it is related to Reese's procedure with the conditions established before.

All programs provide a very accurate maximum moment estimate, as shown in Figure 6.6, even the ones that analyzed the pile as a linear material. It is very difficult to make a distinction. Elastic solutions also provide an accurate maximum moment until 20 kips. This load-maximum moment curve is almost linear until cracking. Then the maximum moment rate increases until 'failure', about 12500 kips-in. The inclinometer p-y curves predict lower maximum moments for a given load than the strain gage p-y curves, due to the latter having lower resistances. However, the difference is not considerable.

The normalized bending moments along the pile increase until pile cracking and are almost constant after load increment #5 i.e. 50 kips. Also, the maximum moment level descends from 150 inches to 210 inches below the load level at loads #1 and #5, respectively. Figure B.4 shows a schematic view of the cracks after loading. Notice that the bigger and wider crack (crack #7) is at 236" from the pile top or 218" from the load level. Unfortunately, the maximum moments were not measured, they were calculate with polynomial regression and three strain gage levels.

All pre-driving in situ p-y curves provide a good estimate of the maximum bending moment per load. The range of maximum moments for any p-y curve method is in the order of 10% for a given load. They also all exhibit the same tendency explained for the load-deflection behavior, for a given load the maximum bending moment depends on the soil resistance established by the p-y curve especially at shallow depths; e.g. for softer p-y curves, higher maximum moments. Finally, the moments along the pile at 30 kips are presented in Figure 6.10. All present the same variation except the PMT curves which have a larger moments due to the softer pressuremeter modulus, even the reload modulus. This modulus is very sensitive to the loading rate. The PMT p-y curves overestimate the maximum moment by 13% and the other methods are in a 6% range at 30 kips. The deeper layers only influence the shape of the moment curve along the pile depth but not the maximum moment as can be seen for the DMT curves. For higher soil modulus values at the cemented sand layer the moment inflection moves upward

CHAPTER 7

PREDICTION AND EVALUATION OF THE PILE GROUPS SUBJECTED TO LATERAL LOAD

Presented in this chapter is a summary of the results of the static lateral load test on the pile groups and a comparison with the behavior predicted by the programs that consider group effects referred to in Chapter 2. Most programs are based on hybrid methods, i.e. superposition of group effects into the single pile behavior.

It is considered "theoretically" that the same soil conditions are founded in both pile groups. In order to simplify the results, the pile groups are divided into four rows that define the average behavior of the piles within the row. A schematic view of the leading, middle leading, middle trailing, and trailing rows for both pile groups and the distances to the mudline are shown in Figure 7.1. The measured load-deflection curves at frame level for both pile groups are presented in Figure 7.2.

The chapter also includes load-deflection response, p-y multipliers, load-distribution, bending moment along the piles, and load-maximum moment.

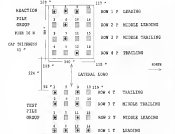


Figure 7.1 Pile Groups Layout.

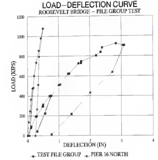


Figure 7.2 Measured Load-Deflection Curves for Pile Groups at Load Level.

7.1 Test Pile Group

The test pile group consisted of sixteen sacrificial piles joined by a steel frame. The pile head was considered free headed against rotation. This group had 10 piles fully instrumented (inclinometer and 9 levels of strain gages) and three piles only with load cells, as shown in Figure 4.5.

7.1.1 Prediction of Load-Deflection Response

The prediction of the load-deflection curves is compared in Figure 7.3 using the programs FLPIER, PIGR3D and GROUP (similar to COM624P for free-head pile) with the 'best' p-y curves with p-y multipliers back calculated from the load test. In this case, the inclinometer p-y curves are chosen because they better represent the single pile response. The average imaginary single pile method is also considered in this figure as an upper limit for deflections. The average measured load-deflection curves for each row are shown in Figure 7.4. The p-y multipliers back calculated from each pile and per row are presented in Figure 7.5. The distance between the load level to the mudline varies from pile to pile. The total group has an average p-y multiplier of 0.55 and an efficiency of 0.8

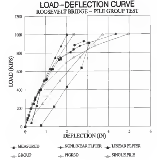


Figure 7-3 Predicted Load-deflection Curves for the Test Pile Group Using Different Programs

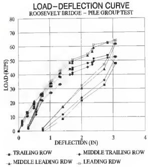


Figure 7.4 Measured Load-Deflection Curves for Each Pile Row in the Test Pile Group.

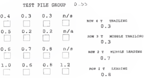


Figure 7.5. P-y Multipliers for the Test Pile Group.

A comparison of the measured load-deflection curve for each measured pile with the predicted using FLPIER and inclinometer p-y curves with a range of p-y multipliers is presented in Figures 7.6 thru 7.9, for the leading, middle leading, middle trailing, and trailing rows, respectively. The average p-y multiplier per row is presented in Figures 7.10 thru 7.13. Finally, Figure 7.14 compares the measured load-deflection curve of the test pile group with the predicted using FLPIER with inclinometer p-y curves and different sets of p-y multipliers.

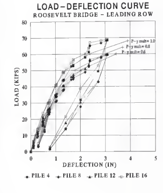


Figure 7.6 Measured and Predicted Load-Deflection Curve for the Leading Row Using FLPIER and a Range of P-y Multipliers

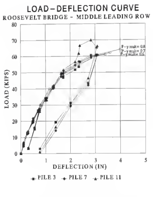


Figure 7-7 Measured and Predicted Load-Deflection Curve for the Middle Leading Row Using FLPIER and a Range of P-y Multipliers

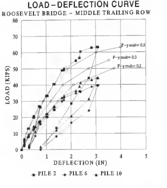


Figure 7.8 Measured and Predicted Load-Deflection Curve for the Middle Trailing Row Using FLPIER and a Range of P-y Multipliers

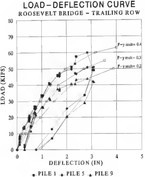


Figure 7.9 Measured and Predicted Load-Deflection Curve for the Trailing Row Using FLPIER and a Range of P-y Multipliers

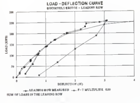


Figure 7.10 P-y Multiplier for Leading Row

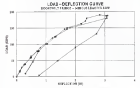


Figure 7.11 P-y Multiplier for Middle Leading Row

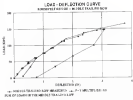


Figure 7.12 P-y Multiplier for Middle Trailing Row.

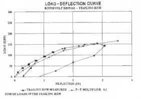


Figure 7 13 P-y Multiplier for Trailing Row

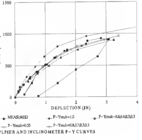


Figure 7 14 Measured and Predicted Load-Deflection for the Test Pile Group Using FLPIER and Different P-y Multipliers

7 1 2 Load-Distribution in the Test Pile Group

The average percentage of load distribution taken by each row during the nine load steps is shown in Figure 7 15. The trailing row takes 31% of the total load during the initial load This decreases to 21% during the final load In contrast, the leading row takes only 22% of the total load during the first load but increases to 28% at the last load. For the middle rows the difference is not so dramatic

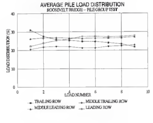


Figure 7 15 Average Load Distribution for each Pile Row

7.1.3 Prediction of Bending Moments

The variation of the maximum measured bending moment with pile head load per row is presented in Figure 7.16. This variation is compared in Figures 7.17 through 7.20 for the instrumented piles of each row, using the program FLPIER with the inclinometer p-y curves and the back calculated p-y multipliers (0.8, 0.7, 0.3, 0.3).

The measured bending moments along the piles for pile #16, pile #3, pile #10, and pile #3 are shown in Figures 7.21 through 7.24. This variation with depth is also compared with the predicted using the program FLPIER.

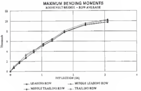


Figure 7.16 Measured Maximum Bending Moment versus Load for Each Pile Row in the Test Pile Group

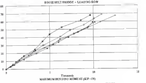


Figure 7-17 Predicted Maximum Bending Moments for the Leading Row Using FLPIER and P-y Multipliers

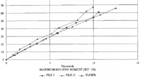


Figure 7-18 Predicted Maximum Bending Moments for the Middle Leading Row Using FLPIER and P-y Multipliers

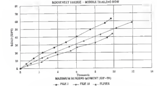


Figure 7.19 Predicted Maximum Bending Moments for the Middle Trailing Row Using FLPIER and P-y Multipliers

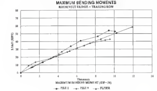


Figure 7.20 Predicted Maximum Bending Moments for the Trailing Row Using FLPIER and P-y Multipliers

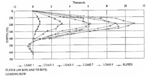


Figure 7 21   Measured Bending Moment Along Pile #16

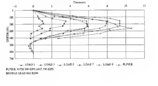


Figure 7.22   Measured Bending Moment Along Pile #3

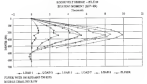


Figure 7.23 Measured Bending Moment Along Pile #10

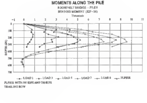


Figure 7.24 Measured Bending Moment Along Pile #8

7.2 Comments on the Pile Group Performance

The piles in the test group began to crack after 750 kips or at 1.8 inches of deformation when the maximum moment reached about 7500 kip-inch in the leading row. The maximum load of the group test was about 924 kips according to the thirteen load cells plus the average row load of the other three. The group had an unrecovered deformation of 0.77 inches at the load frame. At load #4 the group was unloaded to almost zero and reloaded again to the same load.

The pier had a maximum deflection of 0.5 inches at about 1080 kips. The maximum moment reach about 8800 kip-in the leading row, a little higher than the cracking moment, but just for 30 minutes. The pier had an unrecovered deformation of 0.2 inches at the cap. The difference between both loads may be due to friction at the loading frame, accuracy of the load cells, and some rotation of the loading frame in the horizontal and vertical planes. This difference increased until 150 kips (15%) at load 9A and decreased with time (30 minutes) at load 9C. Perhaps longer time steps are needed to reduced this difference.

The average pile group response was softer than the single pile response. However, the single pile test can be a good reference to relate the pile group behavior. The shape of the pile group deflection curve is the same of the single pile with a smaller average load. The p-y multipliers seem to work well to account for this group effect. The group efficiency is about 80% for the initial loads and about 85% for the final load. The pre-driving in situ tests better represent the soil conditions than the post-driving in situ tests for laterally loaded pile groups, even though some densification occurs.

Program FLPIER models very well the load-deflection curve of prestressed piles, especially after pile cracking. GROUP and linear FLPIER provide a good prediction for the initial portion of the curve. They do not account for pile cracking and for pinned head piles and without group interaction, GROUP behaves as COM624P. PIGR3D predicts a very stiff initial portion and lower loads for larger deformations. The group interaction behavior depends on the Young's modulus assumed for the surface layer and the distance to the rigid boundary. Reese's single pile method is just an upper bound of the deflection and is too conservative. Due to the imaginary large diameter, the deflection seems to be a straight line for the loads used.

The average leading row behaved similarly to a single pile. Furthermore, both leading rows and both trailing rows had very similar load-deflection curves. The pile in the leading row took more load than the pile in the trailing. The average leading pile took about 60 kips and the average trailing pile took about 50 kips. Note, that the initial portion of the curves are very similar, the difference arises at larger deformations, more than an inch.

As shown in Figure 7.5, the overall p-y multiplier for the test group is 0.55 and for the leading, middle leading, middle trailing, and trailing row is 0.8, 0.7, 0.3, and 0.3, respectively. There is a reasonable agreement with centrifuge results for 3 D spacing and medium dense sand, for pinned head group the p-y multipliers are 0.8,0.45, and 0.3 and for fixed head group are 0.8,0.4,0.3, and 0.2 (McVay et al. 1995 and 1996b). They also agree with Brown's p-y multipliers 0.8,0.4, and 0.3 (Brown et al., 1988). The pile p-y multipliers were back calculated with the measured lengths between the mudline and loading frame and with the total load per row. Also, the outside piles take more load than inner piles within the row, possibly due to shadow effects. According to the pile driving records the piles where installed in the following pile sequence: 8, 7, 6, 5, 4, 3, 2, 1, 12, 16, 11, 15, 10, 14, 9, and 13. Perhaps, this sequence is the cause of the higher resistance in the outside pile columns and in the second row.

The leading row presents a range of p-y multipliers from 1.0 to 0.6, the middle leading row a very close range from 0.8 to 0.6, the middle trailing row a range from 0.5 to 0.2, and the trailing row a range from 0.4 to 0.2, as shown in Figure 7.6 thru 7.9, respectively. An average p-y multiplier per row that match the sum of each individual measured pile is presented in Figures 7.10 thru 7.13. The values that better match the ultimate soil resistance and the initial portion of the load-deflection curve were found to be 0.8,0.7,0.3, and 0.3 with a sum of 2.1. The variation of the total group response is presented in Figure 7.14. The first curve represents no pile interaction or p-y multipliers of 1.0, the second curve was computed using a overall p-y multiplier of 0.55, the third curve using 0.8,0.7,0.3, and 0.3, and the last curve using 0.8,0.4,0.3, and 0.3.

The problem of using prestressed piles in the test pile group is that the pile rigidity (EI) is not well know and it varies dramatically after cracking. This results in difficultly in finding the ultimate soil resistances for the p-y curves. Also, the concrete Young's modulus increases with time especially if the pile is submerged in water. According to the strain gage readings and the measured load the pile rigidity should be about 3.3 10^8 to 3.5 10^8 kips-inch². This may effect the results but not dramatically, furthermore they are very reasonable.

The load distribution of the trailing row decreased with load, while the leading row increased with load. This change is higher for the initial loads. The load distribution of the middle rows is approximately constant.

The maximum bending moments for the leading row were higher than for the trailing rows. However, all were within a

15% range These moments look very similar if they are plotted with deformation The maximum bending moments ranged between 11000 kip-inch in the leading row and 9500 kip-inch in the trailing row. FLPIER agrees well with the measured moments as shown in Figures 7.17 through 7.20 The differences are due to accuracy of the strain gages and to the fact that the p-y multiplier utilized for the row is an average. The maximum bending moment was at a depth of about 110 to 120 inches for the leading rows, 150 inches for the middle trailing row and about 180 inches for the trailing row FLPIER predicts very well the shape and the depth of the moments along the pile

CHAPTER 8
CONCLUSIONS AND RECOMMENDATIONS

8.1 Conclusions

A lateral load test program was performed in cohesionless soil on a single pile and on a 16 pile group at Roosevelt bridge. As a result of the investigation, the following conclusions can be drawn:

1) The average pile group response was softer than the single pile response. However, the single pile test can be a good indicator of the pile group behavior. The pile group load-deflection efficiency was 80%. For all the rows, the initial portions of the curve were very similar, however, the differences arose at larger deformations. The average leading row pile behaved similarly to a single pile and took more load than the average pile in the trailing rows. Also, outside piles took more load than inner piles within a row, possibly due to a shadow effect and pile driving sequence.

2) The p-y multipliers seem to work well to account for the group effect. The pre-driving in situ tests better provide the soil conditions for laterally loaded pile groups. They have

reasonable agreement with McVay's centrifuge results and Brown's pile group. The overall p-y multiplier for the group was 0.55 for the test pile group and 0.8, 0.7, 0.3, 0.3 for the leading, middle leading, middle trailing, and trailing row, respectively.

3) The pile test group deformed elastically to 750 kips or 1.8 inches of deformation when the maximum moment reached about 7500 kip-inch in the leading row. The maximum load of the group test was about 924 kips. The group had an unrecovered deformation of 0.77 inches at the load frame. The maximum bending moments for the leading row were higher than for the trailing rows. However, all were within a 15% range. The maximum bending moments ranged between 11000 kip-inch in the leading row and 9500 kip-inch in the trailing row. The maximum moments were deeper in the trailing piles.

4) The pier had a maximum deflection of 0.5 inches at about 1080 kips. The maximum moment reached about 8800 kip-inch in the leading row and 6300 kip-inch in the trailing rows. The pier had an unrecovered deformation of 0.2 inches at the cap surface.

5) In the group test the load distribution of the trailing row decreased during the load test, while the leading row increased. This is very dramatic for the first load. The load distribution of the middle rows was approximately constant. In the pier, the load distribution appeared to be constant, too.

6) During the single pile test, the pile began to deform plastically after 50 kips i.e. after pile cracking, about 7500 kip-inch. The maximum load of the lateral load test was about 72 kips and the maximum bending moment about 12500 kips-inch. The maximum moment was located between 150 and 210 inches below the load level, deeper for higher loads. After extracting the failed pile, the biggest pile crack was observed 218 inches below the load level.

7) The Winkler solution, which models the soil response as non-linear springs, seems to be a very reasonable approach for modeling a single pile with the correct p-y curves. The assumption of a linear elastic medium can underestimate pile deflection and bending moments especially for higher load levels with larger deformations.

8) For a laterally loaded single pile without pile-soil-pile interaction, the programs COM624P, FLPIER, GROUP, PIGR3D predict very similarly for linear elastic pile properties. However, only the FLPIER nonlinear option was capable of predicting pile deflection after cracking for these prestressed piles. All computer programs provide similar maximum bending moments and moment distribution. Moments show small sensitivity to these methods, even after cracking.

9) Reese et al. recommendations for p-y curves in sandy layers are very reasonable for the calculation of ultimate soil resistance if the friction angle has been correlated

directly from SPT N values. However, it appears that the coefficient of subgrade reaction is very conservative. A value between 60 and 90 pci was obtained from the single pile test. A suggested SPT p-y curve method can provide a good approximation due to the fact that it is related to Reese's procedure with higher coefficients of subgrade reaction.

10) Dilatometer tests in some way duplicate the load sequence induced in the soil by a driven pile, especially with very small deformation. Thus, the dilatometer modulus (E_D) might be correlated directly to the initial portion of a p-y curve or the modulus of subgrade reaction. However, the ultimate soil resistance depends directly on the angle of internal friction and can be very unconservative, especially if the ϕ angle is derived directly from the dilatometer test.

11) P-y curves developed from PMT results have the advantage of an accurate evaluation of the ultimate soil resistance especially for the Robertson et al. (1984) method. This resistance can be related directly to the pressure limit (P_L). Cone pressuremeter modulus, even from the reload portion of the PMT curve, is too small to be related to the modulus of subgrade reaction.

12) The slope inclinometer method provides a reasonable approach to compute p-y curves for the upper layer to a depth of 120 inches (4 D). For deeper p-y curves the accuracy decreases, however, the first 4 D are the most significantly

for pile-head movements.

13) The p-y curves obtained from the single pile test, present for the surface layer (from 0-160 in.), a very stiff initial slope i.e., between 1200 to 5000 psi and an ultimate soil resistance between 300 and 1500 lbs/in. In the p-y curves obtained from the pile group, the initial slope does not change as significantly as the ultimate soil resistance. The initial slope varies from 600 to 4000 psi in all rows and the ultimate soil resistance varies from 300 to 1000 lbs/in in the leading row and from 100 to 300 lbs/in in the trailing row.

14) Based on in situ tests performed prior to and after pile driving at Pier 16 North, as well as CAPWAP and PDA results, despite jetting soil densification occurred due to soil displaced by pile intrusion, leading to higher soil resistance. In this case, the coefficient of subgrade reaction for the sand layer increased by 100 %, the friction angle from 32° to 40°, and the relative density increased from 55% to 90% approximately. For the cemented sand layer the increment is much smaller. Soil properties between pre-driving and post-driving in situ tests can best represent the axially loaded pile group behavior accordingly to initial shear modulus obtained from instrumented piles in the pier. T-z curves can be related to pre-driving in situ soil tests by multiplying soil parameters by a coefficient due to densification, in this case between 1.5 and 2 for the sandy layer.

8.2 Recommendations

The following recommendations are made based on this investigation:

1) To make the strain gage configuration more versatile, after concrete pouring, the terminal strip should be outside of the pile. Thus, quarter and half bridge configuration can be used if some gages do not work. Also, it seems that welded strain gages are more reliable. They can be welded to round steel bar cages that are more economical than steel pipes.

Strain gage resistances should be checked after installation, after grouting within the pile, and prior to performing the load test. Also a load calibration of the instrumented steel pipe must be performed after gage installation.

2) The inclinometer method utilized to back calculate p-y curves can be improved, in order to have a reliable and economic method to obtain p-y curves rather than back calculate from strain gages. At least a four decimal instrument and 12 meter depth must be required.

3) Cracking bending moment should be a design criteria as well as pile displacement for prestressed piles. It can be computed only with the pile section, axial load, and the prestressed properties.

4) A pile cross sleeve of only 4 to 5 D length below the mudline can be driven after pile driving in order to increase front area and the lateral resistance of the piles. However, the group effect maybe increase.

5) According to the p-y curves obtained from strain gages and inclinometer readings, p-y multipliers should be used only for the ultimate soil resistance, not the coefficient of subgrade reaction. However, this difference seems to be negligible in the load deflection curve.

6) A new single load test pile at Pier 16 North vicinity must be performed to confirm the results presented and to obtain more information from the strain gages along the pile.

7) The pressuremeter probe should be adapted to the electric cone penetrometer to obtain soil data simultaneously in one boring log. This is very valuable when the installation conditions are difficult and time consuming as they were in Pier 16 North. The reload modulus of the PMT should be correlated to the DMT and standardized in order to obtain reliable initial modulus, especially if a protector ring in the tip is used.

8) It is important to measured the force in all the piles of the group test, thus a load cell should be installed in piles even though they are not instrumented.

9) More in situ tests within pile groups should be performed in order to recommend an appropriate densification coefficient

for the axial behavior, especially for loose and medium dense sands

10) In situ tests such as PMT, CPT, DMT, and the SPT (at every 2.5 feet for the first 4 D) can be very reliable tools for obtaining load transfer curves directly. Maybe a spreadsheet or a parallel program can create the input data for the soil layers for FLPIER using in situ soil parameters.

11) Steel pipe piles with constant EI can be more reliable than prestressed piles for computing p-y curves and ultimate soil resistances especially after cracking. Concrete properties are not well defined.

12) Additional field and centrifuge tests with different pile group configurations should be required in order to establish a reliable procedure for closely-spaced pile groups.

APPENDIX A

IN SITU SOIL TESTS AT PROJECT SITE

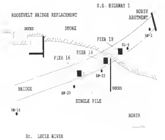


Figure A.1 Roosevelt Bridge Replacement - Plan View

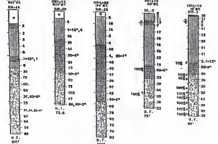


Figure A.2 Soil Profile and SPT Boring Logs

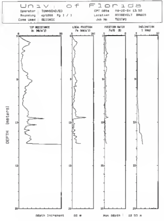


Figure A.3 Cone Penetration Test Profile at Pier 19 (Test #1).

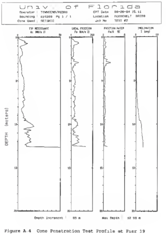


Figure A.4 Cone Penetration Test Profile at Pier 19 (Test #2)

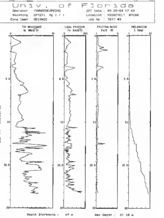


Figure A 5 Cone Penetration Test Profile at North Abutment (Test #3)

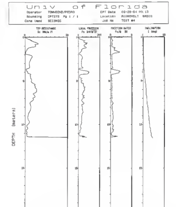


Figure A.6 Cone Penetration Test Profile at North Abutment (Test #4)

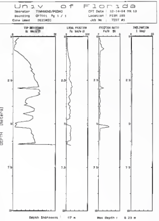


Figure A 7 Cone Penetration Test Profile at Pier 18 South (Test #1)

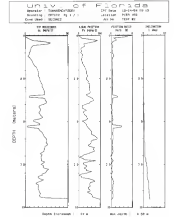


Figure A.8 Cone Penetration Test Profile at Pier 18 South (Test #2)

PIER 19 NORTH – PRE-DRIVING IN SITU TEST
DILATOMETER TEST
DMT-19

SET 27 1994

F T & F.R.

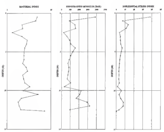
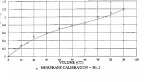
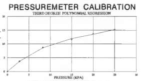


Figure A 9 Dilatometer Test Profile at Pier 19 (Test #1)

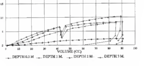

PRESSUREMETER CALIBRATION

THIRD DEGREE POLYNOMIAL REGRESSION


PRESSUREMETER CALIBRATION

THIRD DEGREE POLYNOMIAL REGRESSION

Figure A 10 Pressuremeter Calibration Curves.



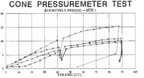
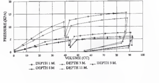


Figure A 11 Pressuremeter Corrected Curves at Pier 19.



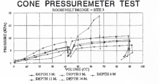


Figure A 12 Pressuremeter Corrected Curves at North Abutment

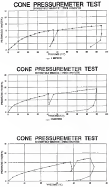
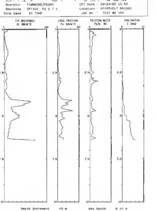


Figure A.13 Pressuremeter Corrected Curves at Pier 18 South



Univ of Florida

Operator TOWNSEND/PEDRO
Sounding CPT182 Pg 1 / 1
Cone Used 10 TONE

CPT Date : 08-23-85 15:59
Location : ROOSEVELT BRIDGE
Job No : TEST #2 16M

Figure A 14 Cone Penetration Test at Vicinity of Pier 16 North

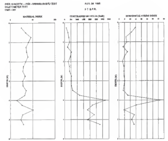


Figure A.15 Dilatometer Test Profile at Pier 16 North
(Test #1)

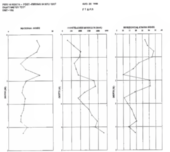


Figure A 16 Dilatometer Test Profile at Pier 16 North (Test #2)

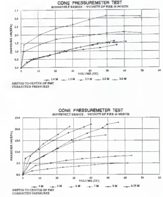


Figure A.17 PMT Corrected Curves Before Pile Driving at Vicinity of Pier 16 North.

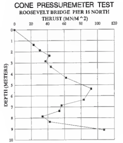


Figure A.18 PMT #1 Thrust Resistance (Vicinity of Pier 16 North).

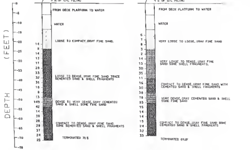

BORING NO. 1 (P-18)
9' E OF E/P FOOTER
9' S. OF C/L PILING
BORING NO 2 (P 18)
20' E OF E/P FOOTER
9' S OF S/L PILING
DEPTH (FEET)
0
-5
-10
-15
-20
-25
-30
-35
-40
-45
-50
-55
-60
-65
-70
-75
FROM DECK PLATFORM TO WATER
WATER
LOOSE TO COMPACT GRAY FINE SAND.
LOOSE TO DENSE, GRAY FINE SAND TRACE CEMENTED SAND & SHELL FRAGMENTS
DENSE TO VERY DENSE, GRAY CEMENTED SAND & SHELL SOME FINE SAND
COMPACT TO DENSE, GRAY FINE SAND SOME CEMENTED SAND & SHELL FRAGMENTS
TERMINATED 71.5
14
8
14
13
17
28
33
32
31
18
8
13
149
42
14
30
26
29
37
24
25
FROM DECK PLATFORM TO WATER
WATER
VERY LOOSE TO LOOSE, GRAY FINE SAND
VERY LOOSE TO DENSE, GRAY FINE SAND SOME SHELL FRAGMENTS.
COMPACT TO DENSE, GRAY FINE SAND WITH CEMENTED SAND & SHELL FRAGMENTS
VERY DENSE, GRAY CEMENTED SAND & SHELL SOME FINE SAND
COMPACT TO DENSE, GRAY FINE SAND SOME CEMENTED SAND & SHELL FRAGMENTS
TERMINATED 69.8'
6
7
8
3
2
14
13
30
35
11
16
30
38
78
93
15
14
14
15
24
32
33

Figure A 19 Soil Profile and SPT Boring Log for Vicinity of Pier 16 North

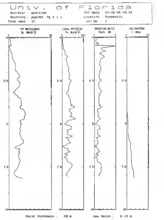
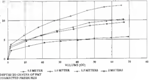


Figure A.20 Cone Penetration Test after Pile Driving at Pier 16 North



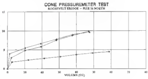


Figure A.21 PMT Corrected Curves after Pile Driving at Pier 16 North.

Table A.1 Cone Penetration Test Interpretation at Pier 19 (Test #1)

```
Operator   TOWNSEND/ER                    CPT Date   *9 26 94 13 56        Hogentogler & Co Inc
On Site Loc ROOSEVELT BRIDG               Cone Used  [illegible]
Job No     [illegible]                    Water table (meters)  2 *
Tot Unit Wt (avg)   17 kN/m^3

  DEPTH            Qc (avg)    Fs (avg)      Rf (avg)     Sigv'       SOIL BEHAVIOUR TYPE       Eq   Dr     PHI     SPT      Su
 (meters)  (feet)  (MN/m 2)    (kN/m 2)        (%)        (MPa)                                      (%)    deg      N      KPa

                                                                     silty sand to sandy silt
                                                                       sand to silty sand
                                                                             sand
                                                                             sand
                                                                           undefined

Dr   All sands (Jamiolkowski et al  1985)      PHI -     Robertson and Campanella 1983       S=  Nk= 10
```

Table A 2 Cone Penetration Test Interpretation at Pier 19 (Test #2)

Operator [illegible]
CPT Site Loc [illegible]
Job No TEST #2
Tot Unit Wt (avg) 17 kN/m 3

CPT Date 08 26 86 15:13
Cone Used [illegible]
Water table (meters) 2

DEPTH (meters)	DEPTH (feet)	Qc (avg) (MN/m 2)	Fs (avg) (kN/m 2)	Rf (avg) (%)	SIGV' (MPa)	SOIL BEHAVIOUR TYPE	Eq Dr (%)	PHI deg	SPT N	Su MPa
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

Dr : All sands (Jamiolkowski et al 1985)   PHI : Robertson and Campanella 1983   Su: Nkt 15

Table A 3 Cone Penetration Test Interpretation at North Abutment (Test #3)

Table A 3 Continued

Operator [illegible] CPT Date [illegible]
On Site Loc [illegible] BRIDGE Cone Used [illegible]
Job No [illegible] Water table (meters) 2.0
Tot Unit Wt (avg) 19 kN/m³

Hogentogler & Co. Inc.

DEPTH (meters)	(feet)	Qc (avg) (MN/m²)	Fs (avg) (kN/m²)	Rf (avg) (%)	SIGV' (kPa)	SOIL BEHAVIOUR TYPE	Eq - Dr (%)	PHI deg	SPT N	Su kPa
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	gravelly sand to sand	[illegible]	[illegible]	[illegible]	UNDEFINED
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	sand	[illegible]	[illegible]	[illegible]	UNDEFINED
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	sand	[illegible]	[illegible]	[illegible]	UNDEFINED
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	sand	[illegible]	[illegible]	[illegible]	UNDEFINED
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	sand	[illegible]	[illegible]	[illegible]	UNDEFINED
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	gravelly sand to sand	[illegible]	[illegible]	[illegible]	UNDEFINED
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	gravelly sand to sand	[illegible]	[illegible]	[illegible]	UNDEFINED
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	gravelly sand to sand	[illegible]	[illegible]	[illegible]	UNDEFINED
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	sand	[illegible]	[illegible]	[illegible]	UNDEFINED
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	sand	[illegible]	[illegible]	[illegible]	UNDEFINED
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	gravelly sand to sand	[illegible]	[illegible]	[illegible]	UNDEFINED
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	sand	[illegible]	[illegible]	[illegible]	UNDEFINED
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	sand	[illegible]	[illegible]	[illegible]	UNDEFINED
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	sand	[illegible]	[illegible]	[illegible]	UNDEFINED
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	gravelly sand to sand	[illegible]	[illegible]	[illegible]	UNDEFINED
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	gravelly sand to sand	[illegible]	[illegible]	[illegible]	UNDEFINED
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	gravelly sand to sand	[illegible]	[illegible]	[illegible]	UNDEFINED
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	gravelly sand to sand	[illegible]	[illegible]	[illegible]	UNDEFINED
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	gravelly sand to sand	[illegible]	[illegible]	[illegible]	UNDEFINED
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	sand	[illegible]	[illegible]	[illegible]	UNDEFINED
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	sand	[illegible]	[illegible]	[illegible]	UNDEFINED
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	sand	[illegible]	[illegible]	[illegible]	UNDEFINED
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	gravelly sand to sand	[illegible]	[illegible]	[illegible]	UNDEFINED
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	sand	[illegible]	[illegible]	[illegible]	UNDEFINED
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	sand	[illegible]	[illegible]	[illegible]	UNDEFINED
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	gravelly sand to sand	[illegible]	[illegible]	[illegible]	UNDEFINED
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	gravelly sand to sand	[illegible]	[illegible]	[illegible]	UNDEFINED
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	gravelly sand to sand	[illegible]	[illegible]	[illegible]	UNDEFINED
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	gravelly sand to sand	[illegible]	[illegible]	[illegible]	UNDEFINED
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	sand	[illegible]	[illegible]	[illegible]	UNDEFINED

Dr All sands (Jamiolkowski et al 1985) PHI Robertson and Campanella 1983 Su Nkt: 15

Table A.4 Cone Penetration Test Interpretation at North Abutment (Test #4)

Operator [illegible] CPT Date [illegible]
On Site Loc [illegible] Cone Used [illegible]
Job No TEST #4 Water table (meters) 1.0
Tot Unit Wt (avg) [illegible] [illegible]

DEPTH (meters)	(feet)	Qc (avg)	Fs (avg)	Rf (avg) (%)	SIGV'	SOIL BEHAVIOUR TYPE	Eq - Dr (%)	PHI deg	SPT N	Su
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	silty sand to sandy silt	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	sand to silty sand	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	sand	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	sand	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	sand to silty sand	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	sandy silt to clayey silt	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	silty clay to clay	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	sandy silt to clayey silt	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	sand to silty sand	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	sand to silty sand	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	sand to silty sand	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	sand to silty sand	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	sand to silty sand	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	sand to silty sand	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	silty sand to sandy silt	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	clayey silt to silty clay	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	silty sand to sandy silt	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	sand to silty sand	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	sand to silty sand	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	sand to silty sand	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	sand to silty sand	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	sand	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	sand	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	sand	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	sand	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	sand	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	sand	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	sand	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	sand	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	sand to silty sand	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	sand to silty sand	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	sand to silty sand	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	sand to silty sand	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	sand to silty sand	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	silty sand to sandy silt	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	silty sand to sandy silt	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	silty sand to sandy silt	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	silty sand to sandy silt	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	sand to silty sand	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	silty sand to sandy silt	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	sandy silt to clayey silt	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	silty sand to sandy silt	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	sandy silt to clayey silt	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	sandy silt to clayey silt	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	silty sand to sandy silt	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	undefined	[illegible]	[illegible]	[illegible]	[illegible]

Dr - All sands (Jamiolkowski et al [illegible]) PHI - Robertson and Campanella 198[illegible] Su: Nk= [illegible]

Table A.5 Cone Penetration Test Interpretation at Pier 18 South (Test #1)

Operator: [illegible]
Cn Site Loc: PIER 18S
Job No: TEST #1
Tot Unit Wt (avg): 20 kN/m^3

CPT Date: 11:14:94 09:11
Cone Used: [illegible]
Water table (meters): 0

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DEPTH (meters)	(feet)	Qc (avg) (MN/m^2)	Fs (avg) (kN/m^2)	Rf (avg) (%)	SIGV' (bar)	SOIL BEHAVIOUR TYPE	Eq	Dr (%)	PHI deg	SPT N	Su bar
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	sand to silty sand		>90	>48	[illegible]	UNDEFINED
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	sandy silt to clayey silt		UNDEFND	UNDEF	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	sand to silty sand		[illegible]	[illegible]	[illegible]	UNDEFINED
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	sand		[illegible]	[illegible]	[illegible]	UNDEFINED
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	sand		[illegible]	[illegible]	[illegible]	UNDEFINED
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	sand		[illegible]	[illegible]	[illegible]	UNDEFINED
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	gravelly sand to sand		[illegible]	[illegible]	[illegible]	UNDEFINED
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	gravelly sand to sand		[illegible]	[illegible]	[illegible]	UNDEFINED
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	gravelly sand to sand		[illegible]	[illegible]	[illegible]	UNDEFINED
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	gravelly sand to sand		[illegible]	[illegible]	[illegible]	UNDEFINED
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	sand		[illegible]	[illegible]	[illegible]	UNDEFINED
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	sand		[illegible]	[illegible]	[illegible]	UNDEFINED
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	sand		[illegible]	[illegible]	[illegible]	UNDEFINED
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	sand		[illegible]	[illegible]	[illegible]	UNDEFINED
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	sand to silty sand		[illegible]	[illegible]	[illegible]	UNDEFINED
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	sand		[illegible]	[illegible]	[illegible]	UNDEFINED
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	gravelly sand to sand		>90	[illegible]	[illegible]	UNDEFINED

Dr - All sands (Jamiolkowski et al 1985)   PHI - Robertson and Campanella 1983   Su: Nkt= 15

Table A.6 Cone Penetration Test Interpretation at Pier 1S South (Test #2)

Operator [illegible]　　　　CPT Date 12 16 94 08 13
On Site Loc: Pier 1S　　　　Cone Used [illegible]
Job No: TEST #2　　　　Water table (meters) [illegible]
Tot. Unit Wt. (avg) 20 kN/m^3

DEPTH (metres)	(feet)	Qc (avg) (MN/m^2)	Fs (avg) (kN/m^2)	Rf (avg) (%)	SIGV' (bar)	SOIL BEHAVIOUR TYPE	Eq Dr (%)	PHI deg	SPT N	Su bar
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	sandy silt to clayey silt	UNDFND	UNDFND	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	sand	[illegible]	[illegible]	[illegible]	UNDEFINED
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	sand to silty sand	[illegible]	[illegible]	[illegible]	UNDEFINED
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	silty sand to sandy silt	[illegible]	[illegible]	[illegible]	UNDEFINED
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	sand to silty sand	[illegible]	[illegible]	[illegible]	UNDEFINED
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	sand	[illegible]	[illegible]	[illegible]	UNDEFINED
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	sand	[illegible]	[illegible]	[illegible]	UNDEFINED
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	sand	[illegible]	[illegible]	[illegible]	UNDEFINED
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	sand	[illegible]	[illegible]	[illegible]	UNDEFINED
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	sand	[illegible]	[illegible]	[illegible]	UNDEFINED
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	sand	[illegible]	[illegible]	[illegible]	UNDEFINED
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	sand	[illegible]	[illegible]	[illegible]	UNDEFINED
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	sand	[illegible]	[illegible]	[illegible]	UNDEFINED
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	sand	[illegible]	[illegible]	[illegible]	UNDEFINED
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	sand	[illegible]	[illegible]	[illegible]	UNDEFINED
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	sand to silty sand	[illegible]	[illegible]	[illegible]	UNDEFINED
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	sand	[illegible]	[illegible]	[illegible]	UNDEFINED
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	sand	[illegible]	[illegible]	[illegible]	UNDEFINED
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	sand	[illegible]	[illegible]	[illegible]	UNDEFINED
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	sand	[illegible]	[illegible]	[illegible]	UNDEFINED
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	sand	[illegible]	[illegible]	[illegible]	UNDEFINED
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	sand	[illegible]	[illegible]	[illegible]	UNDEFINED
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	gravelly sand to sand	[illegible]	[illegible]	[illegible]	UNDEFINED
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	sand	[illegible]	[illegible]	[illegible]	UNDEFINED
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	sand	[illegible]	[illegible]	[illegible]	UNDEFINED
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	sand	[illegible]	[illegible]	[illegible]	UNDEFINED
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	sand	[illegible]	[illegible]	[illegible]	UNDEFINED

Dr　All sands (Jamiolkowski et al 1985)　　PHI - Robertson and Campanella 1983　　Su Nk= 15

Table A.7 Dilatometer Geotechnical Parameters at Pier 19 (Test #1).

```
UNIVERSITY OF FLORIDA GEOTECHNICAL GROUP                       TEST NO  TEST [illegible]
FILE NAME       [illegible]
FILE NUMBER     [illegible]

[illegible]

LOCATION:  [illegible]
[illegible]

CALIBRATION INFORMATION
DELTA A =  [illegible]
[illegible]

[illegible]

[illegible]

END OF SOUNDING
```

Table A.8 Cone Penetration Test Interpretation at Vicinity of Pier 16 North

DEPTH (meters)	(feet)	Qc (avg) (MN/m^2)	Fs (avg) (kN/m^2)	RF (avg) (%)	SIGV (MPa)	SOIL BEHAVIOUR TYPE	R_{c} (%)	Dr	PHI deg	SPT N	Su kPa
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	sand to silty sand	[illegible]	[illegible]	[illegible]	[illegible]	UNDEFINED
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	sensitive fine grained	UNDFND	UNDFD	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	silty sand to sandy silt	[illegible]	[illegible]	[illegible]	[illegible]	UNDEFINED
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	sand to silty sand	[illegible]	[illegible]	[illegible]	[illegible]	UNDEFINED
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	sand to silty sand	[illegible]	[illegible]	[illegible]	[illegible]	UNDEFINED
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	sand	[illegible]	[illegible]	[illegible]	[illegible]	UNDEFINED
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	sand to silty sand	[illegible]	[illegible]	[illegible]	[illegible]	UNDEFINED
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	silty sand to sandy silt	[illegible]	[illegible]	[illegible]	[illegible]	UNDEFINED
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	silty sand to sandy silt	[illegible]	[illegible]	[illegible]	[illegible]	UNDEFINED
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	silty sand to sandy silt	[illegible]	[illegible]	[illegible]	[illegible]	UNDEFINED
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	sand to silty sand	[illegible]	[illegible]	[illegible]	[illegible]	UNDEFINED
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	sand to silty sand	[illegible]	[illegible]	[illegible]	[illegible]	UNDEFINED
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	sand to silty sand	[illegible]	[illegible]	[illegible]	[illegible]	UNDEFINED
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	sand to silty sand	[illegible]	[illegible]	[illegible]	[illegible]	UNDEFINED
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	sand to silty sand	[illegible]	[illegible]	[illegible]	[illegible]	UNDEFINED
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	sand	[illegible]	[illegible]	[illegible]	[illegible]	UNDEFINED
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	gravelly sand to sand	[illegible]	[illegible]	[illegible]	[illegible]	UNDEFINED
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	gravelly sand to sand	[illegible]	[illegible]	[illegible]	[illegible]	UNDEFINED
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	sand	[illegible]	[illegible]	[illegible]	[illegible]	UNDEFINED
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	gravelly sand to sand	[illegible]	[illegible]	[illegible]	[illegible]	UNDEFINED
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	gravelly sand to sand	[illegible]	[illegible]	[illegible]	[illegible]	UNDEFINED
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	sand	[illegible]	[illegible]	[illegible]	[illegible]	UNDEFINED
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	sand	[illegible]	[illegible]	[illegible]	[illegible]	UNDEFINED
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	sand to silty sand	[illegible]	[illegible]	[illegible]	[illegible]	UNDEFINED
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	sand to silty sand	[illegible]	[illegible]	[illegible]	[illegible]	UNDEFINED

Dr All sands (Jamiolkowski et al 1985) PHI : Robertson and Campanella 1983 Su: Nk= 15

Table A.9 Dilatometer Pre-Driving Geotechnical Parameters

```
UNIVERSITY OF FLORIDA           D M T  : F.I. D P D.   24 AUG 1995          Reg 101

ROOSEVELT BRIDGE                PIER 16 SPAIN VICINITY
DMT 161                         10 FEET EAST OF PIER 16 B

Water head  = 1.95

Reduction formulae according to Marchetti  ASCE Geot Jnl ,Mar  1980, Vol 106  299 321
NOTE : OCR = ' relative OCR '  OCR below often reasonable  Accuracy can be improved if precise
       OCR values are available  Then factorize all OCR below by the ratio  OCRreference/OCR

Po    = Corrected A reading              bar          INTERPRETED GEOTECHNICAL PARAMETERS
P1    = Corrected B reading              bar          *
Gamma = Bulk unit weight/GammaH2O        ( )          Ko = In situ earth press. coeff.        (-)
Sigma'= Effective overb. stress          bar          Ocr= Overconsolidation ratio            ( )
U     = Pore pressure                    bar          q  = Eroded overburden above g.e        bar
Id    = Material index                   ( )          M  = Constrained modulus (at Sigma')    bar
Kd    = Horizontal stress index          ( )          Cu = Undrained shear strength           bar
Ed    = Dilatometer modulus              bar

 Z (m)   Po    P1  Gamma  Sigma    U    Id    Kd    Ed   Ko   Ocr    q     M    Cu     DESCRIPTION

  0.25   0.5   1.3  1.70  0.03  0.22  3.30  13.8   29                    68          SAND
  0.50   0.6   2.5  1.70  0.04  0.24  4.08  10.7   46                   108          SAND
  0.75   0.8   3.3  1.70  0.05  0.26  6.46   8.8   95                   194          SAND
  1.00   0.8   3.9  1.70  0.07  0.29  6.69   8.7  104                   236          SAND
  1.25   0.8   2.3  1.70  0.08  0.31  5.32   3.8   50                    94          SAND
  1.50   1.1   5.3  1.70  0.10  0.34  5.44   7.4  146                   326          SAND
  1.75   0.8   4.5  1.70  0.12  0.36  8.18   4.8  124                   250          SAND
  2.00   0.8   2.8  1.70  0.14  0.39  4.97   3.8   73                   107          SAND
  2.25   1.1   3.7  1.70  0.15  0.41  3.90   4.3   81                   181          SAND
  2.50   1.2   3.9  1.70  0.17  0.44  3.87   4.3   95                   187          SAND
  2.75   1.3   5.3  1.80  0.18  0.46  4.65   4.5  138                   251          SAND
  3.00   1.5   5.3  1.80  0.21  0.49  3.62   5.0  131                   249          SAND
  3.25   1.5   5.3  1.80  0.23  0.51  3.71   4.4  131                   236          SAND
  3.50   1.5   6.3  1.80  0.25  0.53  5.11   3.8  188                   276          SAND
  3.75   2.7  11.8  1.80  0.27  0.56  4.68   7.0  321                   735          SAND
  4.00  11.1  31.8  2.15  0.30  0.58  1.87  34.3  721                  2683          SILTY SAND
  4.25   5.5  21.9  2.00  0.32  0.61  3.33  15.4  568                  1655          SAND
  4.50   2.1  18.1  1.80  0.34  0.63 10.52   4.4  554                   992          SAND
  4.75   5.7  25.3  2.00  0.37  0.66  3.52  13.7  612                  1713          SAND
  5.00   5.1  18.1  2.00  0.39  0.68  2.95  11.3  452                  1182          SILTY SAND
  5.25   6.3  27.0  2.00  0.41  0.71  3.68  13.4  731                  2087          SAND

 Z (m)   Po    P1  Gamma  Sigma'   U    Id    Kd    Ed   Ko   Ocr    q     M    Cu     DESCRIPTION
```

Table A 10 Cone Penetration Test Interpretation after Pile Driving at Pier 16 North

Hogentogler & Co. Inc

Operator Pedro /Ed
On Site Loc Roosevelt
Job No [illegible]
Tot. Unit Wt. (avg) = kN/m^3

CPT Date 04-18 96 08 11
Cone Used 10
Water table (meters) 0

DEPTH (meters)	(feet)	Qc (avg) (MN/m^2)	Fs (avg) (kN/m^2)	Rf (avg) (%)	SIGV' (kPa)	SOIL BEHAVIOUR TYPE	Eq (%)	Dr	PHI deg	SPT N	Su kPa
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	silty sand to sandy silt	[illegible]		[illegible]	[illegible]	UNDEFINED
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	silty sand to sandy silt	[illegible]		[illegible]	[illegible]	UNDEFINED
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	sandy silt to clayey silt	UNDEFINED		UNDEFINED	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	sand to silty sand	[illegible]		[illegible]	[illegible]	UNDEFINED
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	sand to silty sand	[illegible]		[illegible]	[illegible]	UNDEFINED
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	sand	[illegible]		[illegible]	[illegible]	UNDEFINED
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	sand	[illegible]		[illegible]	[illegible]	UNDEFINED
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	sand	[illegible]		[illegible]	[illegible]	UNDEFINED
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	sand	[illegible]		[illegible]	[illegible]	UNDEFINED
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	sand	[illegible]		[illegible]	[illegible]	UNDEFINED
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	sand	[illegible]		[illegible]	[illegible]	UNDEFINED
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	sand	[illegible]		[illegible]	[illegible]	UNDEFINED
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	sand to silty sand	[illegible]	[illegible]	[illegible]	[illegible]	UNDEFINED
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	sand	[illegible]		[illegible]	[illegible]	UNDEFINED
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	sand	[illegible]		[illegible]	[illegible]	UNDEFINED
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	sand	[illegible]		[illegible]	[illegible]	UNDEFINED
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	gravelly sand to sand	[illegible]		[illegible]	[illegible]	UNDEFINED
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	gravelly sand to sand	[illegible]		[illegible]	[illegible]	UNDEFINED
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	gravelly sand to sand	[illegible]		[illegible]	[illegible]	UNDEFINED
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	gravelly sand to sand	[illegible]		[illegible]	[illegible]	UNDEFINED
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	sand	[illegible]	[illegible]	[illegible]	[illegible]	UNDEFINED
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	sand	[illegible]	[illegible]	[illegible]	[illegible]	UNDEFINED
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	sand	[illegible]	[illegible]	[illegible]	[illegible]	UNDEFINED
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	gravelly sand to sand	[illegible]	[illegible]	[illegible]	[illegible]	UNDEFINED
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	gravelly sand to sand	[illegible]	[illegible]	[illegible]	[illegible]	UNDEFINED
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	sand	[illegible]	[illegible]	[illegible]	[illegible]	UNDEFINED
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	sand	[illegible]	[illegible]	[illegible]	[illegible]	UNDEFINED
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	sand	[illegible]	[illegible]	[illegible]	[illegible]	UNDEFINED
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	sand	[illegible]	[illegible]	[illegible]	[illegible]	UNDEFINED
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	sand	[illegible]	[illegible]	[illegible]	[illegible]	UNDEFINED
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	sand	[illegible]	[illegible]	[illegible]	[illegible]	UNDEFINED

Dr All sands (Jamiolkowski et al 1985) PHI Robertson and Campanella 1983 Su Nk= 15

Table A.11 Dilatometer Post-Driving Geotechnical Parameters.

```
UNIVERSITY OF FLORIDA            D M T - F C. & P R. - 25 AUG 1995               Reg 162

ROOSEVELT BRIDGE                 PIER 16 NORTH CAP (NE CORNER)
DMT 162                          PILE CAP 16 NORTH (NE CORNER)

Water head = 1.00

Reduction formulae according to Marchetti, ASCE Geot Jnl ,Mar 1980, Vol 106, 299-321
NOTE : OCR = 'relative OCR'. OCR below often reasonable. Accuracy can be improved if precise
       OCR values are available. Then factorize all OCR below by the ratio OCRreference/OCR

Po     = Corrected A reading          bar     INTERPRETED GEOTECHNICAL PARAMETERS
P1     = Corrected B reading          bar
Gamma  = Bulk unit weight/GammaH2O    (-)     Ko = In situ earth press. coeff       (-)
Sigma'= Effective overb. stress       bar     Ocr= Overconsolidation ratio          (-)
U      = Pore pressure                bar     q  = Eroded overburden above p.a      bar
Id     = Material Index               (-)     M  = Constrained modulus (at Sigma')  bar
Kd     = Horizontal stress index      (-)     Cu = Undrained shear strength         bar
Ed     = Dilatometer modulus          bar

 Z (m)   Po    P1  Gamma  Sigma'  U     Id    Kd    Ed    Ko    Ocr   q    M    Cu    DESCRIPTION

  0.25   0.7   2.9  1.70  0.00  0.22  4.55  26.1   76                    258        SAND
  0.50   1.3   6.5  1.80  0.04  0.24  5.17  28.4  181                    633        SAND
  0.75   2.1  10.5  1.80  0.08  0.26  4.42  33.5  283                   1033        SAND
  1.00   2.8   9.5  1.80  0.09  0.29  2.09  32.5  232                    839        SILTY SAND
  1.25   2.5   6.7  1.80  0.10  0.31  1.91  22.0  145                    471        SILTY SAND
  1.50   2.5   7.7  1.80  0.12  0.34  2.48  17.0  181                    552        SILTY SAND
  1.75   1.8   8.6  1.80  0.14  0.36  5.23   9.9  247                    616        SAND
  2.00   5.5  17.7  2.00  0.16  0.39  2.51  50.6  429                   1580        SILTY SAND
  2.25   6.1  18.5  2.00  0.19  0.41  2.17  50.0  429                   1550        SILTY SAND
  2.50   4.4  12.5  1.90  0.21  0.44  2.33  19.0  280                    869        SILTY SAND
  2.75   2.9   8.8  1.90  0.23  0.46  2.38  10.6  203                    520        SILTY SAND
  3.00   3.8  11.5  1.90  0.25  0.49  2.20  15.2  265                    733        SILTY SAND
  3.25   5.0  14.6  2.00  0.28  0.51  2.10  16.4  342                   1014        SILTY SAND
  3.50   6.0  16.5  2.00  0.30  0.53  1.92  18.2  364                   1115        SILTY SAND
  3.75   7.1  19.5  2.00  0.32  0.56  1.88  20.2  429                   1358        SILTY SAND
  4.00   5.5  16.3  2.00  0.35  0.58  2.61  14.0  444                   1252        SILTY SAND
  4.25   7.8  25.1  2.00  0.37  0.61  2.14  19.2  531                   1655        SILTY SAND

 Z (m)   Po    P1  Gamma  Sigma'  U     Id    Kd    Ed    Ko    Ocr   q    M    Cu    DESCRIPTION
```

APPENDIX B

SINGLE PILE LOAD TEST

Table B-1   Single Pile Test Load-Deflection Record.

LOAD NUMBER	TIME	GAUGE PRESSURE PSI	LOAD CELL READING	LOAD KIPS	DEFLECTION LVDT (IN)	DEFLECTION AT LOAD (IN)
0	01:18 PM	0	-1000	0.0	0.00	0.00
1	01:28 PM	1300	66	7.3	0.00	0.00
2	01:43 PM	2250	915	17.9	0.24	0.22
3	02:53 PM	3450	2005	26.2	0.48	0.46
4	03:02 PM	4600	3005	35.9	0.76	0.72
5	03:35 PM	5800	4015	45.6	1.06	1.08
6	03:52 PM	7100	5000	56.0	1.86	1.82
7*	04:57 PM	8250	**	64.5	2.87	2.68
8*	05:30 PM	9000	**	71.4	4.22	4.00
9	06:43 PM	6000	**	47.2	2.97	3.78
10	06:50 PM	3750	**	28.0	2.90	3.65
11	06:55 PM	2000	**	15.0	2.88	2.57
12	06:58 PM	0	**	0.0	1.24	1.18

* Jack setup changed to increase stroke.

** Load cell did not work.

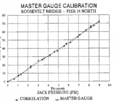


Figure B-1   Jack Calibration.

Table B 2 Load-Deflection Readings

LOAD #	READING #	POT 2 (VOLTS)	POT 3 (VOLTS)	DEFLECTION 1 (IN)	DEFLECTION 2 (IN)	DEFLECTION (IN)
1	1	-0.707	-0.009	0.0000	0.0000	0.0001
2	1	-0.706	-0.202	0.0000	0.0000	
	2	-3.796	-0.305	0.0000	0.0000	
	3	-3.796	-0.402	0.0000	1.0000	
	3	-3.796	-3.302	0.0000	1.0000	0.0000
3	2	-3.866	-3.145	3.7300	3.7750	
	3	-1.366	-3.157	3.1920	3.7000	
	3	3.077	-3.107	0.2087	0.2101	
	4	-1.565	-3.780	0.2087	0.2091	0.2101
2	1	-3.570	-0.043	0.4277	0.2901	
	3	8.164	-0.313	3.8607	0.494	
	3	-3.047	-3.310	3.4801	0.446	
	3	-3.309	-3.067	3.4861	0.4965	0.0000
4		0.648	-3.910	3.0752	0.0000	
	9	3.000	-7.300	3.7320	5.0000	
	2	-0.427	-3.836	3.7000	0.7148	
	7	-3.611	-7.667	3.7720	0.7448	3.0000
5	1	-0.258	-2.755	0.8880	3.0462	
	2	-3.968	3.727	2.0004	1.0129	
	2	-3.368	-2.707	2.0004	1.0024	
	2	-3.356	7.000	2.0004	1.0300	7.0004
7	1	-0.300	7.073	1.4304	1.8000	
	3	-3.693	-7.300	2.0004	1.8300	
	3	-7.004	-7.330	7.0004	1.8304	
	6	7.300	7.304	7.7004	1.0300	7.0004
7	2	7.460	0.221	2.0047	3.0300	
	3	-7.320	3.813	0.2087	3.0000	
	3	7.477	3.794	0.0820	0.0000	
	4	-7.300	3.730	0.0000	5.0000	3.8713
7		-3.300	-3.130	4.2181	4.7057	
	3	-3.977	3.117	3.2087	4.8121	
	2	-3.970	-0.196	3.3937	4.8121	
	3	-6.973	3.700	7.3917	4.8182	0.0000
9	7	-6.799	-0.067	3.9779	0.0000	
	2	-3.750	3.857	3.0779	0.0000	0.0000
10	1	-7.028	3.760	3.0000	3.0000	
	2	-7.306	-3.760	0.1029	3.0000	3.0000
11	1	-2.454	-5.886	3.7107	3.0797	
	3	7.434	-3.000	3.7107	3.9797	2.0047
12	1	3.550	-7.412	1.2863	1.0004	
	3	-3.000	-7.913	7.0004	1.0004	1.0063

Table B.3 Single Pile Strain Gage Readings

STRAIN GAGE SHEET

FILE No: [illegible]
DATE: [illegible]
OPERATOR: [illegible]

ROOSEVELT BRIDGE PIER 11 SOUTH
UNIVERSITY OF FLORIDA

PAGE No: 1

STRAIN GAGE LEVEL	DEPTH TO PILE TOP (FT)	STRAIN AT EACH STRAIN GAGE LEVEL (MICROSTRAINS)							
		LOAD NUMBER 1				LOAD NUMBER 2			
		1 min	5 min	10 min	15 min	1 min	5 min	10 min	15 min
1	[illegible]								
2	[illegible]								
3	[illegible]								
4	[illegible]	−6	−6	7	−7	−34	30	−30	−34
5	[illegible]								
6	[illegible]	−30	30	10	−12	−17	17	−27	−36
7	[illegible]								
8	[illegible]	−18	17	16	−12	30	−40	−47	−8
9	[illegible]	−17	−11	16	17	30	−54	−58	−57

STRAIN GAGE LEVEL	DEPTH TO PILE TOP (FT)	STRAIN AT EACH STRAIN GAGE LEVEL (MICROSTRAINS)							
		LOAD NUMBER 3				LOAD NUMBER 4			
		1 min	5 min	10 min	15 min	1 min	5 min	10 min	15 min
1	[illegible]								
2	[illegible]								
3	[illegible]								
4	[illegible]	30	−94	−56	30	70	−30	−66	−90
5	[illegible]								
6	[illegible]	30	−30	−50	30	30	−80	−84	−66
7	[illegible]								
8	[illegible]	−34	−66	−66	−66	30	−91	−94	30
9	[illegible]	34	−30	−30	30	−66	−77	−30	75

STRAIN GAGE LEVEL	DEPTH TO PILE TOP (FT)	STRAIN AT EACH STRAIN GAGE LEVEL (MICROSTRAINS)							
		LOAD NUMBER 5				LOAD NUMBER 6			
		1 min	5 min	10 min	15 min	1 min	5 min	10 min	15 min
1	[illegible]								
2	[illegible]								
3	[illegible]								
4	[illegible]	117	−118	−120	−108	−110	100	−104	−148
5	[illegible]								
6	[illegible]	−73	71	−90	−95	−140	−100	−108	124
7	[illegible]								
8	[illegible]	112	114	−118	−18	172	172	−180	186
9	[illegible]	80	80	−86	80	130	130	−137	−136

Table B 3 Continued

STRAIN GAGE SHEET

FILE No: SINGLE PILE
DATE: [illegible]
OPERATOR: [illegible]

ROOSEVELT BRIDGE PIER 16 NORTH
UNIVERSITY OF FLORIDA

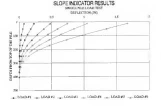
DATE No. 1

STRAIN GAGE LEVEL	DEPTH TO PILE TOP (IN)	STRAIN AT EACH STRAIN GAGE LEVEL (MICROSTRAIN)							
		LOAD NUMBER 7				LOAD NUMBER 8			
		1 MIN	5 MIN	10 MIN	15 MIN	1 MIN	5 MIN	10 MIN	15 MIN
1	[illegible]								
2	[illegible]								
3	[illegible]								
4	[illegible]	380	381	−380	−382	−405	−404	−411	−410
5	[illegible]								
6	[illegible]	−475	−475	443	−480	−734	758	−750	−750
7	[illegible]								
8	[illegible]	−500	380	−383	381	380	−132	671	−880
9	[illegible]	102	−211	353	353	−890	−368	353	−880

STRAIN GAGE LEVEL	DEPTH TO PILE TOP (IN)	STRAIN AT EACH STRAIN GAGE LEVEL (MICROSTRAIN)							
		LOAD NUMBER 9		LOAD NUMBER 10		LOAD NUMBER 11		LOAD NUMBER 12	
		1 MIN	5 MIN	1 MIN	5 MIN	1 MIN	5 MIN	1 MIN	5 MIN
1	[illegible]								
2	[illegible]								
3	[illegible]								
4	[illegible]	−427	−427	381	−381	−363	−363	−143	−141
5	[illegible]								
6	[illegible]	−7 5	735	−402	−402	−308	308	−50	−57
7	[illegible]								
8	[illegible]	−480	−480	−380	752	211	−211	77	−77
9	[illegible]	−481	−481	567	−540	−133	−133	38	38

Note: [illegible]

APPROXIMATE LOAD STEP 50 KIPS
TIME INTERVAL BETWEEN LOADS 15 MINUTES
No OF READINGS FOR LOAD STEP 4
READINGS RESULTS WERE TAKEN AT 1, 5, 10, AND 15 MINUTES



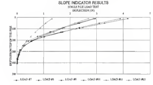


Figure B-2 Measured Deflections Along the Pile.

Table B 4   Single Pile Inclinometer Readings

SLOPE INCLINOMETER DEFLECTION SHEET

FILE No: SINGLE PILE ROOSEVELT BRIDGE PIER 16 NORTH PAGE No. 1
DATE 07 Nov-95 UNIVERSITY OF FLORIDA
OPERATOR: J. Anderson

SLOPE INDICATOR LEVEL	DEPTH TO PILE TOP (IN)	DEFLECTION AT SLOPE INCLINOMETER LEVEL (INCHES)							
		Deflection at load level:				Deflection at load level:			
		LOAD NUMBER 1			0.0956	LOAD NUMBER 2			0.3163
		Initial	Reading	Slope	Deflection (in)	Initial	Reading	Slope	Deflection (in)
1	10	−9.56	10.07	0.000130	0.0736	−9.56	9.8	0.000130	0.0116
2	51	13.28	13.80	0.000080	0.0621	−13.32	−14.28	0.000088	0.1784
3	91	−14.79	15.18	0.000070	0.1028	−14.79	−16.07	0.000080	0.1503
4	130	11.74	10.80	0.000160	0.0081	−11.74	−11.64	0.000080	0.0884
5	170	−0.26	−0.45	0.000130	0.0086	−0.26	−0.75	0.000090	0.0106
6	209	−0.30	−0.36	0.000030	0.0050	−0.30	0.80	0.000080	0.0206
7	248	2.87	−2.90	0.000040	0.0004	−2.87	0.46	0.000070	0.0188
8	289	−0.36	−0.38	0.000030	0.0001	−0.36	−0.36	0.000080	0.0118
9	327	5.95	5.91	0.000080	0.0004	5.95	5.95	0.000000	−0.3010
10	369	8.15	8.15	0.000000	0.0006	8.15	8.17	−0.000010	−0.3010
11	405	3.71	−3.71	0.000070	0.0000	−3.72	−3.70	−0.000020	−0.0006

SLOPE INCLINOMETER DEFLECTION SHEET

FILE No: SINGLE PILE ROOSEVELT BRIDGE PIER 16 NORTH
DATE 07 Nov-95 UNIVERSITY OF FLORIDA
OPERATOR: J. Anderson

SLOPE INDICATOR LEVEL	DEPTH TO PILE TOP (IN)	DEFLECTION AT SLOPE INCLINOMETER LEVEL (INCHES)							
		Deflection at load level:				Deflection at load level:			
		LOAD NUMBER 3			0.4620	LOAD NUMBER 4			0.7811
		Initial	Reading	Slope	Deflection (in)	Initial	Reading	Slope	Deflection (in)
1	10	−9.56	−12.09	0.001530	0.6780	−9.56	−13.34	0.003780	0.7417
2	51	−13.31	−15.04	0.001650	0.3760	−13.31	−17.08	0.003760	0.5959
3	91	−14.79	−16.98	0.001570	0.2791	−14.79	−18.06	0.003070	0.4866
4	130	−11.74	−13.41	0.001670	0.1907	−11.74	−14.08	0.002880	0.3181
5	170	−0.26	1.64	0.001890	0.1201	0.26	2.43	0.002080	0.2016
6	207	−0.80	1.40	0.000880	0.0864	−0.80	2.01	0.001810	0.1113
7	198	−2.87	−3.61	0.000550	0.0357	2.87	3.83	0.000880	0.0616
8	287	−0.35	−0.34	0.000080	0.0118	0.36	−0.18	0.000480	0.0186
9	327	5.95	5.87	0.000080	0.0086	5.95	5.96	0.000080	0.0041
10	366	8.15	8.11	−0.000020	−0.0016	8.15	8.17	−0.000020	−0.0008
11	405	3.71	−3.70	−0.000020	−0.0003	3.72	3.68	−0.000020	−0.0008

Table B 4 Continued.

SLOPE INCLINOMETER DEFLECTION SHEET

PILE No: SINGLE PILE
DATE: 07-Nov-95
OPERATOR: [illegible]

ROOSEVELT BRIDGE PIER 16 NORTH PAGE No. 2
UNIVERSITY OF FLORIDA

SLOPE INDICATOR LEVEL	DEPTH TO PILE TOP (IN)	DEFLECTION AT SLOPE INCLINOMETER LEVEL (INCHES)							
		Deflection at load level: LOAD NUMBER 5 [illegible]				Deflection at load level: LOAD NUMBER 6 [illegible]			
		Initial	Reading	Slope	Deflection (in)	Initial	Reading	Slope	Deflection (in)
1	12	−9.56	10.77	0.006220	1.0604	5.96	15.76	0.006940	1.6477
2	51	19.52	−18.47	0.009960	0.8601	−13.33	−21.45	0.008120	1.3441
3	91	−14.75	−19.54	0.004550	0.8889	−14.70	−21.24	0.005940	1.0310
4	130	−11.74	−15.81	0.004940	0.8050	−11.74	−16.41	0.004670	0.7505
5	170	−0.34	5.34	0.005710	0.6491	−0.34	−5.26	0.004940	0.6491
6	209	−0.50	−2.07	0.002170	1.0489	0.00	3.77	0.003270	0.8159
7	248	−7.07	−4.21	0.004940	0.3712	2.07	−4.91	0.002840	0.1669
8	287	−0.26	1.06	0.004940	0.0000	−0.20	−1.33	0.001070	0.0480
9	327	5.05	5.00	0.002500	0.0136	5.05	5.43	0.000320	0.0220
10	366	8.16	8.11	0.000040	0.0097	8.15	8.00	0.000040	0.0090
11	405	−3.70	−3.05	−0.000040	−0.0016	−3.71	−3.40	−0.000040	−0.0038

SLOPE INCLINOMETER DEFLECTION SHEET

PILE No: SINGLE PILE
DATE: 07-Nov-95
OPERATOR: [illegible]

ROOSEVELT BRIDGE PIER 16 NORTH
UNIVERSITY OF FLORIDA

SLOPE INDICATOR LEVEL	DEPTH TO PILE TOP (IN)	DEFLECTION AT SLOPE INCLINOMETER LEVEL (INCHES)							
		Deflection at load level: LOAD NUMBER 7 [illegible]				Deflection at load level: LOAD NUMBER 8 [illegible]			
		Initial	Reading	Slope	Deflection (in)	Initial	Reading	Slope	Deflection (in)
1	12	−9.56	24.60	0.011070	2.0609	−9.56	−34.52	0.021960	4.1609
2	51	−18.52	−27.94	0.010670	2.5580	−13.30	−34.33	0.004940	4.0903
3	91	−14.76	28.26	0.012960	1.7750	−14.79	−34.18	0.019050	0.8889
4	130	−11.74	30.69	0.012920	1.2405	−11.74	−30.86	0.017190	1.7084
5	170	−0.34	−9.35	0.008880	0.7730	−0.36	10.73	0.012460	1.0800
6	209	−0.50	8.73	0.009210	0.6211	−0.50	−7.50	0.004940	0.5331
7	248	2.87	3.30	0.003920	0.2137	−2.07	−6.60	0.003750	0.2776
8	287	−0.26	1.81	0.001030	1.1109	−0.28	2.51	0.004940	1.1207
9	327	5.91	9.95	0.003750	0.0611	2.07	8.84	0.001100	0.0897
10	366	8.15	7.90	0.000040	0.0000	8.15	7.64	0.000010	0.0140
11	405	−3.71	−3.72	0.000040	0.0000	−3.72	3.01	0.000040	0.0053

Table B 4 Continued

SLOPE INCLINOMETER DEFLECTION SHEET

FILE No: SINGLE PILE
DATE 09-Nov-95
OPERATOR: I. Assoum

ROOSEVELT BRIDGE PIER 10 NORTH PAGE No 3
UNIVERSITY OF FLORIDA

SLOPE INDICATOR LEVEL	DEPTH TO PILE TOP (in)	DEFLECTION AT SLOPE INCLINOMETER LEVEL (INCHES)							
		Deflection at load level: LOAD NUMBER 9			3.7739	Deflection at load level: LOAD NUMBER 10			2.7916
		Initial	Reading	Slope	Deflection (in)	Initial	Reading	Slope	Deflection (in)
1	10	−8.56	29.60	0.095600	3.8839	−8.56	28.17	0.014510	0.0700
2	38	−13.32	−33.02	0.119190	3.1610	13.00	16.96	0.115640	0.0000
3	90	−14.79	30.12	0.117750	2.3498	18.79	−14.46	0.006250	1.7600
4	150	−11.74	−37.64	0.116100	1.6510	11.74	36.76	0.018460	1.3870
5	170	−8.56	13.26	0.111930	1.0177	−8.56	9.96	0.009600	0.0000
6	200	−0.10	−7.10	0.008000	0.0000	−0.10	−6.76	0.005900	0.5420
7	300	−3.87	−4.66	0.003900	0.2772	−3.87	0.10	0.000800	0.0000
8	307	−0.28	−2.10	0.009640	0.1280	−0.10	2.20	0.000000	0.1150
9	307	3.85	4.60	0.001070	0.0000	3.85	4.85	0.001000	0.0000
10	346	8.15	7.80	0.000170	0.0104	8.15	7.75	0.008800	0.0015
11	405	−3.72	−3.70	0.000007	0.0050	3.79	−3.86	0.000100	0.0050

SLOPE INCLINOMETER DEFLECTION SHEET

FILE No: SINGLE PILE
DATE 07-Nov-95
OPERATOR: I. Assoum

ROOSEVELT BRIDGE PIER 10 NORTH
UNIVERSITY OF FLORIDA

SLOPE INDICATOR LEVEL	DEPTH TO PILE TOP (in)	DEFLECTION AT SLOPE INCLINOMETER LEVEL (INCHES)							
		Deflection at load level: LOAD NUMBER 11			2.4416	Deflection at load level: LOAD NUMBER 12			0.0000
		Initial	Reading	Slope	Deflection (in)	Initial	Reading	Slope	Deflection (in)
1	10	−8.56	35.50	0.012940	2.4917	8.56	15.09	0.006130	1.5177
2	87	13.32	30.52	0.019000	4.0820	−13.32	−18.33	0.009010	1.0764
3	91	−14.79	−29.00	0.014710	3.7100	−14.79	19.21	0.004720	0.8791
4	180	−11.74	−31.21	0.010470	1.2744	−11.74	13.20	0.008600	0.6950
5	170	−8.56	−8.55	0.008610	0.8080	−8.56	−4.43	0.008000	0.5126
6	200	−0.30	6.56	0.005840	0.5232	−0.30	−5.53	0.003080	0.5536
7	240	−3.87	−0.38	0.003710	0.2983	3.87	−5.13	0.003260	0.2135
8	280	−0.10	2.30	0.000600	0.1490	−0.30	−5.85	0.003370	0.1445
9	327	5.85	6.88	0.003370	0.0000	3.88	6.88	0.003370	0.0800
10	346	8.15	7.70	0.000870	0.0107	8.15	7.47	0.000800	0.0175
11	405	−3.72	−3.95	0.000130	0.0050	3.72	−4.87	0.000130	0.0179

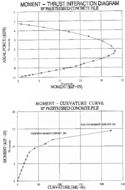
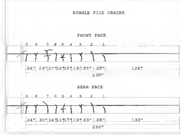


Figure B-3 Prestressed Concrete Pile Bending Moments



Cracks Dimensions

	Size	Dimension
1	.004"	14"
2	.006"	9"
3	.004"	17"
4	.004"	15"
5	.010"	10"
6	.005"	16"
7	.030"	24"
8	.004"	17"
9	.001"	12"

Figure B 4 Single Piles Cracks after Loading (Figgs Construction Services)

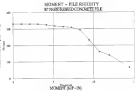


Figure B 5 Bending Moment-Pile Rigidity Curve

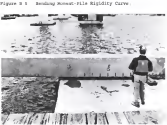


Figure B 6 Photograph of Pile Cracks After Loading

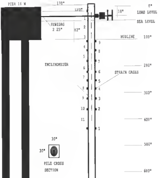


Figure 3.7 Single Pile Load Test Instrumentation Layout

APPENDIX C

SACRIFICIAL PILE GROUP LOAD TEST

Table C.1 Horizontal Displacements Readings

LOAD NUMBER	TIME (HH:MM)	WIRE LINE # 1 (BW) READING (IN)	WIRE LINE # 1 (BW) HORIZONTAL DISPLACEMENT (IN)	WIRE LINE # 2 (ST) READING (IN)	WIRE LINE # 2 (ST) HORIZONTAL DISPLACEMENT (IN)	WIRE LINE # 3 (NW) READING (IN)	WIRE LINE # 3 (NW) HORIZONTAL DISPLACEMENT (IN)	WIRE LINE # 4 (SE) READING (IN)	WIRE LINE # 4 (SE) HORIZONTAL DISPLACEMENT (IN)
0 A	11:00 AM	1.000	0.000	1.000	0.000	0.000	0.000	0.000	0.000
0 B	11:08 AM	1.000	0.000	1.000	0.000	1.000	0.000	0.000	0.000
1 A	11:10 AM	1.100	0.125	1.100	0.100	1.000	[illegible]	1.000	1.000
1 B	11:20 AM	1.140	[illegible]	1.100	0.100	1.000	[illegible]	1.000	1.000
1 C	11:42 AM	1.140	[illegible]	1.100	0.100	1.000	[illegible]	1.000	0.110
2 A	11:48 AM	1.000	0.000	1.030	0.010	1.150	2.000	1.000	0.000
2 B	11:58 AM	1.000	0.000	1.000	0.000	2.000	0.000	1.000	1.000
2 C	12:18 PM	1.330	0.330	1.000	0.000	2.000	2.000	1.000	1.000
3 A	12:13 PM	1.000	0.000	1.750	0.440	1.000	2.000	1.000	0.000
3 B	12:21 PM	1.000	0.125	1.750	1.000	1.000	2.000	1.000	0.000
3 C	12:31 PM	1.000	0.125	1.750	1.000	1.000	2.000	1.000	0.000
4 A	12:40 PM	1.750	0.750	1.000	0.000	1.000	0.750	1.000	1.000
4 B	12:50 PM	1.750	0.750	1.700	0.000	1.000	0.750	1.070	0.725
4 C	01:10 PM	1.000	0.000	1.750	0.000	1.000	0.750	1.000	0.750
4 D	01:20 PM	1.000	0.000	1.000	0.000	1.000	0.000	1.000	0.540
4 E	01:00 PM	1.000	0.313	1.240	0.000	1.000	0.750	1.000	0.740
5 A	01:33 PM	0.000	1.000	1.000	0.000	1.000	1.000	1.000	0.070
5 B	01:42 PM	0.000	1.000	2.000	1.000	1.000	1.000	1.000	1.010
5 C	01:56 PM	0.000	1.000	2.000	1.000	1.000	1.000	1.000	1.000
6 A	02:03 PM	0.000	1.000	2.000	1.000	0.000	1.000	0.000	1.000
6 B	02:17 PM	0.000	1.000	2.000	1.000	2.000	1.000	0.000	1.000
6 C	02:28 PM	0.000	1.000	2.000	1.000	0.000	1.000	2.270	1.000
7 A	02:30 PM	0.000	1.000	2.000	1.000	0.000	1.000	0.000	1.000
7 B	02:39 PM	2.000	1.700	2.750	1.000	0.000	1.720	0.020	1.000
7 C	02:48 PM			2.710	1.710				
8 A	02:56 PM	2.000	2.000	2.100	2.100	3.100	2.000	0.100	2.100
8 B	03:06 PM	3.410	3.410	3.340	3.000	0.000	3.000	0.010	3.000
8 C	03:16 PM	0.000	2.000	2.000	2.000	0.000	3.500	2.000	0.000
9 A	03:21 PM	2.000	2.000	2.030	2.000	3.700	0.000	0.700	2.110
9 B	03:31 PM	2.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
9 C	03:44 PM	4.135	3.125	4.115	3.110	0.000	0.000	0.000	0.000
10 A	03:50 PM	0.750	2.750	0.750	2.740	0.000	2.720	0.000	2.000
10 B									
10 C									
11 A	03:56 PM	[illegible]	[illegible]	0.000	0.000	2.000	2.000	0.000	1.070
11 B	03:58 PM	0.000	0.000	0.000	0.000	1.000	0.000	0.000	1.000
11 C	04:10 PM	0.000	0.000	1.000	1.000	1.000	0.000	0.000	1.000
12 A	04:00 PM	0.000	1.750	0.000	0.000	1.000	1.000	0.000	0.000
12 B									
12 C									
13 A	02:05 PM	1.000	1.000	1.770	0.770	1.000	0.780	1.000	2.000
13 B	03:00 PM	1.000	0.000	1.700	2.000	1.000	0.730	1.000	2.000
13 C									

Table C 2   Load Cells and Potentiometers Readings

LOAD #	TOTAL LOAD CELL		JACK PRESSURE		PILE LOAD CELLS			POTENTIOMETERS	
					Pile 0 1	Pile 0 1	Pile 1 1	POT #1	POT #1
	READING	Kips	psi	Kips	Kips	Kips	Kips	in	in
0a	−5000	−8	0	0	0.00	0.00	0.00	0.506	1.300
1a	−330	140	380	135	4.55	0.00	0.66	0.116	0.300
1b	380	142	660	105	3.13	3.32	0.00	0.186	2.307
1c	−380	137	960	111	2.16	2.34	8.85	0.133	5.099
2a	215	990	1175	300	10.50	1.86	11.15	0.500	0.536
2b	760	861	1175	300	18.11	1.80	12.40	0.300	0.383
2c	861	980	1675	300	14.42	0.01	18.71	0.300	0.315
3a	380	372	2600	372	10.78	12.44	46.00	0.375	0.491
3b	616	388	3800	371	10.21	12.16	54.05	0.634	0.300
3c	380	381	3850	373	21.14	14.66	21.66	0.300	0.383
4a	1300	483	3750	300	22.11	11.06	36.00	0.154	0.166
4b	1144	471	3750	300	56.85	17.00	38.15	0.871	2.379
4c	1500	477	3750	300	60.20	11.00	38.60	0.856	0.660
4d	300	380	1175	210	34.10	0.50	10.05	0.730	5.525
5a	−1140	−1	ABOUT 0	0	1.66	1.17	8.50	0.380	5.120
4f	1300	422	3750	300	56.72	15.60	35.30	0.881	0.175
5a	2400	607	4675	603	56.21	60.30	33.80	1.262	1.103
5b	3020	380	4675	322	22.84	60.30	33.90	1.270	1.164
5c	3180	611	4675	563	56.00	20.11	36.25	1.334	1.500
6a	3600	960	3600	750	40.27	34.73	41.60	1.000	1.300
6b	3600	730	3600	750	43.98	34.72	41.00	1.300	1.338
6c	4960	798	3550	730	88.56	34.80	71.60	1.148	1.601
7a	3600	380	3580	172	40.08	56.50	45.80	0.059	1.300
7b	3647	380	3580	373	46.82	35.21	48.20	0.054	2.287
7c	3530	330	9450	561	57.60	82.80	41.15	2.287	2.134
8a	4811	974	7500	1000	42.72	34.79	56.80	2.512	2.333
8b	3500	971	7500	1000	11.56	20.94	56.80	2.370	0.700
8c	4340	380	7380	380	46.67	56.62	58.40	2.305	0.700
9a	3600	1071	3030	1150	56.86	55.75	80.80	0.480	3.271
9b	4480	1380	2050	1150	55.78	56.56	56.00	3.762	0.300
9c	3600	1070	8675	1107	50.06	80.17	56.20	0.924	3.675
10a	3930	751	5750	750	56.75	26.11	45.00	5.237	3.110
11b	1790	996	3750	800	30.43	9.98	46.00	3.380	3.367
12b	300	380	1675	330	19.73	9.95	14.08	1.380	1.790
13b	−1300	12	1	0	0.71	6.00	8.95	1.193	1.060

Table C 3 Strain Gage Readings of Pile 1

BST GROUP PILE

LOAD #	LOAD CELL Kips	STRAIN GAGE READINGS 100 in. ue	160 in. ue	70 in. ue	311 in. ue	347 in. ue	268 in. ue	541 in. ue	358 in. ue	437 in. ue
0a	0.33	0	0	-0	0	0	1	0	0	0
1a	0.80	0	16	32	10	11	4	0	1	0
1b	0.37	11	21	31	20	10	0	0	3	0
1c	7.30	11	22	30	21	10	6	7	4	0
2a	13.60	16	30	32	30	30	16	17	11	0
2b	13.41	17	30	33	30	38	20	23	15	0
2c	18.01	30	60	60	60	33	30	30	15	0
3a	19.00	60	54	70	81	30	30	30	30	0
3b	19.70	27	54	79	84	54	40	41	30	0
3c	20.48	30	60	30	60	54	50	50	29	0
4a	18.05	30	71	101	84	71	61	60	36	0
4b	20.00	30	71	111	85	73	40	57	40	0
4c	26.71	30	71	100	88	71	38	64	38	0
4d	14.54	21	40	54	40	30	50	54	30	0
4e	0.30	0	10	10	20	30	34	30	11	0
4f	18.65	30	70	90	80	60	60	60	30	0
5a	33.77	44	84	120	102	36	60	60	40	0
5b	35.05	44	80	120	134	80	80	81	80	-0
5c	34.70	80	80	121	137	100	88	80	80	0
6a	45.00	54	136	146	146	160	107	100	47	0
6b	40.81	54	134	140	152	131	130	100	80	0
6c	30.85	80	134	140	134	134	129	100	79	0
7a	46.21	80	10	124	148	170	134	112	84	0
7b	46.11	61	102	160	184	200	200	106	100	0
7c	46.60	60	120	184	180	231	183	103	114	0
8a	51.20	81	101	200	204	200	180	180	121	0
8b	53.10	80	107	207	200	330	208	230	130	0
8c	57.80	75	140	300	218	334	274	245	140	0
9a	38.80	73	101	300	341	380	300	300	140	0
9b	45.43	77	103	311	300	437	380	300	130	0
9c	40.11	77	102	310	300	444	371	280	101	0
10a	35.77	91	103	317	300	373	307	273	180	0
10b	25.13	32	80	130	100	300	300	214	134	0
10c	7.30	71	82	78	154	100	160	151	100	0
10d	-0.55	-0	-0	-0	48	78	34	80	49	0

Table C 4   Strain Gage Readings of Pile 2.

TEST GROUP   PILE

LOAD #	LOAD CELL kips	STRAIN GAGE READINGS 140 in. με	108 in. με	175 in. με	211 in. με	247 in. με	283 in. με	319 in. με	355 in. με	427 in. με
0a	0.00	0	0	0	0	0	0	−4	0	−4
1a	5.00	0	9	10	10	0	0	0	0	0
1b	6.33	11	10	10	10	15	11	0	0	0
1c	6.47	11	20	15	11	14	13	0	0	1
2a	13.00	22	20	20	21	20	27	10	13	9
2b	13.26	22	20	22	21	24	30	21	17	8
2c	13.17	25	20	26	21	33	31	27	17	5
3a	20.00	33	40	38	38	30	30	30	27	8
3b	14.81	33	41	47	42	30	31	33	25	10
3c	20.75	34	30	39	34	54	38	36	34	12
4a	28.28	30	34	50	38	47	50	61	50	16
4b	26.77	30	38	61	50	30	50	50	50	16
4c	16.80	11	20	30	71	21	30	30	44	26
4d	11.80	30	31	30	50	30	36	50	30	32
4e	1.00	0	7	0	17	11	30	31	61	30
4f	25.31	30	30	61	72	75	67	38	30	30
5a	30.17	61	30	76	110	30	100	37	50	30
5b	33.33	61	38	79	110	30	105	61	50	30
5c	33.26	61	30	75	116	30	103	94	71	30
6a	40.40	30	30	113	130	114	105	126	30	44
6b	45.00	30	30	116	130	130	100	131	35	30
6c	40.40	30	30	118	130	130	105	130	30	61
7a	46.00	71	110	130	130	172	130	137	112	30
7b	46.40	70	112	130	170	180	180	135	110	31
7c	46.81	71	117	130	172	80	130	137	126	30
8a	30.00	77	105	130	187	200	184	130	130	61
8b	36.85	102	130	200	230	246	240	134	130	30
8c	36.95	116	136	211	200	211	250	181	130	70
9a	50.75	116	112	205	201	207	200	300	130	75
9b	55.17	110	174	230	260	300	340	300	130	75
9c	50.14	114	176	263	300	300	340	208	418	77
10a	46.15	30	140	178	240	320	314	330	320	75
11a	48.40	30	71	107	136	130	340	165	175	30
10b	15.44	84	30	30	30	110	232	165	130	73
1aa	−0.12	−1	1	0	30	30	30	47	76	30

Table C 5   Strain Gage Readings of File 3

TEST GROUP   FILE   3

LOAD #	LOAD CELL Kips	STRAIN GAGE READINGS 100 in. ue	139 in. ue	175 in. ue	211 in. ue	247 in. ue	283 in. ue	319 in. ue	355 in. ue	427 in. ue
00	0.00	0	0	0	30	−3	−4	1	−3	−17
1a	8.70	11	8	12	30	4	−3	8	−8	−8
1b	8.70	11	12	21	30	8	8	8	−5	−3
1c	5.96	11	18	32	34	−4	20	8	−3	−7
2a	12.05	19	27	47	30	84	30	10	5	−61
2b	13.80	30	38	60	61	60	1	64	5	−13
2c	13.40	14	30	46	30	61	11	30	8	8
3a	15.79	24	30	30	80	30	60	50	10	2
3b	20.45	81	44	78	100	78	30	80	10	−6
3c	21.00	38	60	85	102	73	40	60	60	−7
4a	30.79	32	60	103	137	60	88	60	60	−1
4b	38.85	34	60	130	132	32	80	32	60	6
4c	37.30	38	73	108	147	60	81	91	60	−6
4d	13.95	21	61	69	138	81	80	60	60	8
4e	3.00	5	13	19	83	86	80	30	60	10
4f	37.91	41	75	100	121	106	80	68	60	−1
5a	54.13	50	80	137	186	121	108	60	50	10
5b	34.71	60	69	190	160	114	121	30	60	24
5c	34.06	52	95	137	171	86	121	69	73	17
6a	51.59	61	111	180	190	146	130	60	85	60
6b	41.30	52	111	87	202	186	109	102	81	60
6c	40.05	51	110	160	180	188	130	146	102	47
7a	47.00	60	130	108	190	190	130	108	111	70
7b	34.05	47	151	100	301	131	210	190	121	79
7c	17.98	71	145	210	303	300	172	108	86	60
8a	51.98	73	180	130	347	302	305	152	160	60
8b	53.40	85	167	173	452	300	100	181	160	81
8c	38.60	94	80	302	300	284	302	188	135	110
9a	34.08	101	187	302	461	310	461	300	160	79
9b	60.70	87	200	300	461	330	300	300	176	80
9c	60.10	80	180	300	408	300	317	108	180	81
10a	60.60	80	184	302	305	401	300	300	121	130
10b	61.20	80	110	161	160	300	350	171	173	132
10c	18.34	8	74	80	150	108	300	120	180	110
10d	3.64	2	30	80	120	1311	100	64	115	101

Table C 6 Strain Gage Readings of Pile 4

TEST GROUP PILE 4

LOAD #	LOAD CELL Kips	STRAIN GAGE READINGS 128 in. µε	150 in. µε	175 in. µε	211 in. µε	247 in. µε	283 in. µε	319 in. µε	455 in. µε	427 in. µε
0a	0.00	2	1	3	5	1	5	1	6	3
1a	0.80	7	1	5	7	2	2	1	1	3
1b	2.40	10	1	10	1	1	14	4	2	1
1c	0.60	10	1	11	12	1	16	1	5	1
2a	5.05	20	13	24	24	30	31	12	6	1
2b	8.50	24	16	25	30	21	34	15	7	2
2c	8.50	24	16	26	30	21	32	12	8	1
3a	15.35	37	35	30	35	38	101	30	12	2
3b	16.36	38	35	41	44	38	146	36	14	2
3c	17.50	38	35	30	36	48	118	38	18	5
4a	24.02	51	36	36	21	33	179	38	18	2
4b	24.53	51	38	38	35	54	159	38	16	2
4c	25.54	54	36	54	75	58	136	39	30	2
4d	13.55	33	24	38	35	38	134	36	13	2
4e	3.54	3	3	12	31	12	64	36	9	2
4f	30.33	36	16	38	78	38	130	47	31	2
5a	34.75	38	52	30	30	30	388	54	35	11
5b	40.80	38	51	30	30	38	388	54	36	10
5c	35.47	38	55	51	38	44	388	38	35	11
6a	45.60	38	51	33	116	38	385	31	37	14
6b	45.80	38	64	38	115	16	381	31	38	16
6c	44.29	35	88	38	117	38	388	4	35	8
7a	38.45	81	76	118	145	49	485	3	38	35
7b	52.74	88	75	118	153	61	530	−3	37	21
7c	53.21	87	38	116	120	64	588	−6	39	38
8a	38.11	105	38	101	172	75	477	−1	36	27
8b	61.16	130	116	105	208	100	481	10	38	30
8c	38.14	115	120	173	280	109	508	−12	38	38
9a	38.72	116	144	135	380	276	580	−8	38	36
9b	58.58	120	181	307	308	303	510	11	36	31
9c	38.16	184	120	405	388	303	588	−21	38	38
10b	48.88	38	38	110	517	280	405	−38	44	35
11b	31.88	16	77	38	405	107	560	−88	24	16
12b	13.54	38	11	38	138	38	108	−88	5	16
13b	1.40	1	38	13	75	−4	16	−88	−7	2

Table C 7   Strain Gage Readings of Pile 8

TEST GROUP   PILE   1

LOAD #	LOAD CELL Kips	STRAIN GAGE READINGS								
		100 in. με	140 in. με	176 in. με	211 in. με	247 in. με	283 in. με	311 in. με	355 in. με	391 in. με
0a	0.00	1	1	−0	−0	1	1	−22	−1	0
1a	4.11	13	16	12	11	11	1	1	1	1
1b	5.30	13	14	11	10	14	8	1	1	1
1c	6.32	15	16	11	16	16	11	17	3	1
2a	10.00	60	27	60	30	60	32	−10	1	1
2b	10.30	60	68	32	52	24	60	11	11	4
2c	11.02	84	66	60	55	84	60	−11	11	8
3a	16.00	66	11	60	53	55	41	−24	10	7
3b	17.10	80	80	11	45	54	30	11	40	1
3c	17.66	57	80	52	55	56	90	−4	31	1
4a	22.00	88	66	85	34	71	60	60	27	16
4b	26.00	86	60	80	74	71	31	−1	80	11
4c	20.00	81	87	70	80	79	80	60	80	11
4d	13.80	51	61	40	51	60	80	−4	37	11
4e	3.00	7	10	11	19	90	80	−01	51	12
4f	24.40	66	86	56	79	90	71	70	60	17
5a	34.00	90	72	88	100	91	109	−2	62	60
5b	30.00	80	72	60	120	64	107	−3	60	24
5c	34.00	31	72	90	127	66	114	40	90	27
6a	37.47	72	66	130	100	112	100	60	76	31
6b	37.30	73	67	130	147	120	106	60	90	34
6c	36.37	73	68	112	146	125	141	130	80	38
7a	44.20	66	101	140	146	146	100	106	90	38
7b	44.00	11	102	100	137	104	100	80	71	60
7c	60.00	60	100	100	100	107	110	104	79	60
8a	34.00	60	112	177	230	100	180	80	71	60
8b	30.00	110	191	300	211	204	300	211	31	48
8c	30.00	100	144	300	300	300	300	180	40	11
9a	47.00	100	100	270	302	301	308	200	60	60
9b	52.56	104	167	300	406	362	365	212	241	60
9c	50.00	180	248	300	338	360	310	300	200	60
10a	32.00	80	144	333	338	307	300	300	282	60
11a	27.04	57	60	126	180	300	212	130	300	80
12a	14.20	21	40	79	300	130	131	115	100	80
13a	1.00	5	16	60	90	80	60	40	110	80

Table C 8 Strain Gage Readings of Pile 9

TEST GROUP PILE 9

LOAD	LOAD	STRAIN GAGE READINGS								
#	DIAL	100 in.	138 in.	175 in.	211 in.	247 in.	285 in.	310 in.	365 in.	367 in.
	Kips	με	με	με	με	με	με	με	με	με
0a	0.00	0	0	0	0	0	0	0	-0	0
1a	5.02	14	16	18	14	18	8	4	6	0
1b	6.30	17	18	18	18	14	10	1	6	0
1c	8.21	17	18	18	18	13	10	6	6	0
2a	10.27	21	30	34	34	8	26	10	14	0
2b	13.18	30	27	34	35	30	32	26	17	0
2c	19.18	30	27	34	34	35	30	30	11	0
3a	15.32	44	18	36	50	56	53	36	28	0
3b	17.77	53	35	58	70	80	60	52	33	0
3c	16.61	56	36	51	75	56	60	50	33	0
4a	35.32	56	30	75	87	58	75	84	56	0
4b	40.68	86	35	75	87	80	77	90	44	0
4c	40.60	80	80	75	87	84	80	80	36	0
4d	11.38	80	80	80	80	88	73	58	16	0
4e	0.17	1	0	85	81	80	36	88	30	0
4f	35.37	35	86	77	88	87	88	88	35	0
5a	26.40	88	80	88	103	118	115	108	84	0
5b	40.80	88	80	88	105	50	104	116	88	0
5c	40.68	35	80	84	106	112	117	118	88	0
6a	51.80	80	76	113	137	137	137	187	91	0
6b	61.80	80	77	113	137	138	138	187	81	0
6c	61.10	81	77	111	127	141	84	180	87	0
7a	57.80	80	38	135	181	180	188	180	131	0
7b	46.00	89	80	130	184	175	166	88	120	0
7c	57.87	88	87	180	152	177	188	180	180	0
8a	40.62	100	163	134	158	207	211	173	183	0
8b	40.40	150	126	171	210	207	192	211	184	0
8c	40.40	114	137	180	207	184	207	207	189	0
9a	40.90	88	148	205	241	217	334	271	175	0
9b	40.80	135	125	205	241	307	312	305	320	0
9c	41.90	145	138	207	207	331	279	305	307	0
10a	29.06	87	96	176	160	271	307	273	219	0
10b	20.91	81	90	30	131	130	160	180	173	0
10c	7.20	14	30	80	80	80	175	188	130	0
10d	2.16	-1	-0	10	27	44	80	78	32	0

Table C.9 Strain Gage Readings of Pile 10

TEST GROUP PILE : 1

LOAD #	LOAD CELL Kips	STRAIN GAGE READINGS 163 in. με	188 in. με	173 in. με	1 1 in. με	167 in. με	336 in. με	311 in. με	200 in. με	427 in. με
0a	0.00	0	0	0	0	0	0	0	-1	1
1a	3.00	10	10	10	10	6	7	6	1	-3
1b	3.71	12	10	17	10	-4	16	7	1	11
1c	8.00	13	10	17	10	6	11	6	3	-10
2a	8.26	30	33	30	30	30	24	16	9	-41
2b	4.28	31	30	34	34	30	27	22	16	-30
2c	3.80	30	30	34	34	51	37	32	14	-37
3a	12.00	34	44	51	61	70	63	30	21	-35
3b	16.11	30	33	32	30	30	47	41	24	-40
3c	13.00	37	48	30	30	30	33	30	30	-24
4a	17.00	38	60	70	80	84	80	86	34	-36
4b	17.30	38	80	70	80	81	84	80	30	-46
4c	17.77	38	61	80	74	84	80	80	86	-3
4d	15.19	38	80	80	80	71	84	81	84	-8
4e	3.87	0	0	14	16	30	30	30	22	-3
4f	16.09	38	61	73	71	100	72	30	81	-24
5a	20.11	30	30	31	84	120	90	90	88	6
5b	22.16	54	30	30	80	109	80	94	80	6
5c	22.17	30	77	30	86	100	110	88	88	-1
6a	26.88	30	80	112	110	100	121	100	70	-11
6b	28.70	30	80	118	120	180	130	108	90	-17
6c	35.88	61	91	121	193	188	141	108	77	-8
7a	35.91	34	108	188	180	206	165	136	140	18
7b	38.46	64	116	188	165	186	170	136	110	30
7c	38.12	30	118	188	165	206	171	168	116	30
8a	38.39	60	122	172	306	348	190	161	117	60
8b	38.81	39	136	190	275	314	330	306	180	73
8c	40.30	102	139	216	306	349	373	306	188	75
9a	38.16	107	190	306	300	381	380	306	180	80
9b	44.16	108	179	306	306	410	382	306	190	30
9c	46.81	108	178	171	306	413	380	306	182	30
10a	63.25	74	139	139	276	342	306	306	199	84
10b	30.72	31	77	116	190	206	206	181	136	61
10c	15.07	37	41	84	80	130	196	120	108	80
10d	5.70	1	1	16	61	30	61	30	30	27

Table C.10 Strain Gage Readings of Pile 11

TEST GROUP PILE 1

LOAD #	LOAD CELL Kips	STRAIN GAGE READINGS 100 in. ue	148 in. ue	175 in. ue	8?? in. ue	250 in. ue	300 in. ue	111 in. ue	3?? in. ue	437 in. ue
00	0.00	-0	1	36	1	1	1	1	1	-0
1a	1.10	7	1	11	1	11	1	4	1	1
1b	0.12	1	11	30	11	10	1	1	6	1
1c	0.80	1	16	-9	11	11	1	7	1	1
2a	13.30	12	24	30	30	83	51	11	12	1
2b	13.40	11	35	12	35	80	34	11	17	1
2c	13.40	12	35	30	38	45	54	18	19	1
3a	11.60	30	30	30	38	35	38	25	24	1
3b	18.50	31	38	30	35	34	41	33	27	11
3c	26.80	34	18	30	51	54	60	38	27	13
4a	35.30	35	35	36	77	80	64	81	50	19
4b	38.81	38	53	73	77	77	34	36	57	18
4c	38.11	30	33	33	81	38	35	30	46	25
4d	11.10	27	30	30	30	30	41	41	35	13
4e	0.21	1	1	1	10	45	34	36	30	14
4f	26.50	36	53	72	84	85	64	34	44	25
5a	30.80	50	60	87	103	111	34	57	55	34
5b	30.10	30	34	80	105	113	60	38	36	56
5c	38.13	36	33	100	107	106	60	38	71	84
6a	36.40	11	30	105	130	135	60	30	84	32
6b	46.80	33	62	106	104	145	105	36	36	35
6c	11.30	30	50	121	130	146	100	36	30	40
7a	46.01	13	30	136	130	171	106	106	36	50
7b	30.50	71	30	158	180	200	130	144	115	45
7c	51.67	13	34	158	140	201	136	165	112	38
8a	58.30	79	146	130	307	309	105	122	109	46
8b	61.79	90	183	257	380	307	208	309	156	35
8c	66.30	100	207	270	388	302	252	256	150	56
9a	56.28	100	230	280	347	361	300	258	145	37
9b	30.46	87	202	307	330	430	307	347	146	11
9c	34.88	44	200	301	360	301	280	257	140	53
10a	46.45	30	171	301	280	300	287	240	146	81
10b	50.30	42	131	121	173	330	281	180	131	81
10c	16.40	21	81	145	130	187	155	131	130	38
10d	1.05	-1	10	58	82	66	50	35	52	38

Table C.11 Strain Gage Readings of Pile 12

TEST GROUP PILE 12

LOAD #	LOAD (TOTAL) kips	STRAIN GAGE READINGS 130 in. με	155 in. με	178 in. με	211 in. με	247 in. με	280 in. με	511 in. με	105 in. με	427 in. με
0a	0.00	0	0	0	2	0	0	-4	7	-9
1a	4.87	8	10	15	7	14	7	-8	8	0
1b	5.07	10	13	18	14	18	10	9	0	1
1c	5.10	18	18	21	27	21	10	-16	2	1
2a	11.66	20	20	30	53	53	24	-20	7	8
2b	11.46	25	27	44	81	46	28	-27	2	3
2c	12.88	29	37	48	69	58	30	-76	2	2
3a	18.10	36	41	50	106	71	44	56	11	8
3b	18.62	37	42	71	149	85	45	-85	12	8
3c	20.87	39	46	75	137	58	48	-68	11	8
4a	30.30	47	58	86	133	101	50	-64	81	11
4b	30.35	55	56	87	138	105	80	-73	69	11
4c	30.36	48	58	101	180	118	73	-75	81	18
4d	15.07	35	28	48	118	58	60	-86	18	18
4e	8.73	8	8	18	81	37	30	-55	1	18
4f	37.10	46	59	124	137	118	77	-75	27	18
5a	33.74	81	73	125	179	130	88	-58	46	19
5b	33.82	81	74	135	180	136	88	-65	46	28
5c	34.08	86	75	168	172	148	103	-40	51	33
6a	40.46	72	88	188	188	171	103	-31	46	35
6b	40.84	78	90	180	198	186	125	-26	37	37
6c	50.74	76	88	183	205	176	106	-54	71	33
7a	46.16	88	105	144	244	201	165	-30	80	38
7b	46.61	88	108	188	196	207	186	-31	88	38
7c	47.51	88	111	185	274	155	135	-40	86	39
8a	51.06	88	145	196	305	195	179	-31	84	36
8b	51.48	88	156	186	375	394	166	-16	177	81
8c	54.67	105	156	165	165	185	168	-17	131	47
9a	54.88	119	207	304	508	566	305	-88	118	45
9b	56.56	118	220	280	547	447	307	18	116	56
9c	58.74	115	229	885	541	505	314	26	116	56
10a	43.58	77	156	165	506	461	165	10	116	51
11a	35.44	81	86	130	387	185	224	7	116	56
12a	17.77	38	80	38	105	130	140	-55	51	46
13a	2.72	4	81	8	180	88	86	-31	88	86

Table C.12 Strain Gage Readings of Pile 16

TEST GROUP PILE 16

LOAD #	LOAD CELL Kips	STRAIN GAGE READINGS 100 ft με	158 ft με	175 ft με	215 ft με	247 ft με	280 ft με	311 ft με	350 ft με	381 ft με
0S	0.00	0	0	0	0	0	-1	0	0	-0
1a	0.00	8	10	10	11	8	8	8	0	-1
b	4.19	10	16	16	16	10	8	8	8	-0
1c	6.00	11	14	12	17	13	-10	7	5	0
2a	10.75	26	30	26	36	30	8	10	15	1
2b	14.30	30	31	30	40	30	27	13	13	2
2c	14.80	30	3	30	41	33	30	15	10	8
3a	21.80	36	46	36	53	54	30	31	21	0
3b	22.10	37	50	30	56	50	50	56	25	0
3c	22.60	40	54	50	71	61	60	56	31	0
4a	29.61	50	70	84	81	79	70	47	36	7
4b	29.67	50	50	50	80	30	73	49	36	1
4c	30.60	51	30	30	36	81	73	54	36	10
4d	16.47	50	30	84	38	30	50	49	34	8
4e	8.17	7	18	30	30	29	30	34	18	11
4f	31.36	30	51	100	100	81	50	57	61	10
5a	40.75	35	107	120	140	130	30	80	80	19
5b	40.37	36	108	109	148	130	90	71	52	20
5c	41.40	30	100	125	148	130	90	75	57	18
6a	46.30	50	101	140	172	150	100	80	71	18
6b	46.48	50	100	140	177	150	100	80	72	16
6c	48.30	50	140	150	181	170	144	80	70	21
7a	54.67	81	170	180	179	200	181	80	60	24
7b	59.70	80	170	180	200	200	173	115	80	25
7c	46.80	80	150	200	200	240	172	110	80	25
8a	30.30	104	200	252	300	272	180	300	81	30
8b	63.75	114	240	260	300	341	201	130	80	30
8c	67.00	109	200	300	340	301	200	131	80	30
9a	22.45	200	275	300	300	414	300	130	80	30
9b	68.01	200	300	401	370	302	330	100	130	46
9c	66.14	171	300	300	300	473	300	180	136	41
10a	58.75	91	300	300	407	380	308	150	38	27
11b	64.07	80	44	138	300	390	300	144	200	41
12b	61.75	81	88	110	300	179	362	130	80	37
30S	3.80	0	27	42	84	75	38	40	46	31

Table C.13 Slope Inclinometer Readings of Pile 1

PILE No: 7-1
DATE 13-Jun-88
OPERATOR: Paulo Pinto

ROOSEVELT BRIDGE PIER 10 NORTH
UNIVERSITY OF FLORIDA
Depth indicator = Sloo 1520

PAGE No 1
Group Test

SLOPE INDICATOR LEVEL	DEPTH TO LOAD LEVEL (in)	DEFLECTION AT SLOPE INCLINOMETER LEVEL (INCHES)						
		LOAD NUMBER 9A			LOAD NUMBER 30			LOAD 0
		A180	A0	Sum	A180	A0	Sum	0000 Reading
1	0	0.0197	−0.0186	0.0027	0.0000	−0.0188	0.0094	0.0070
2	41	0.0000	−0.0067	0.0000	0.0090	−0.0000	0.0027	0.0000
3	81	0.0100	0.0150	0.0273	0.0110	−0.0150	0.0075	0.0027
4	120	0.0190	−0.0007	0.0041	0.0188	0.0074	0.0000	0.0052
5	159	0.0079	−0.0150	0.0188	0.0074	−0.0150	0.0184	0.0186
6	198	0.0001	−0.0000	0.0191	0.0050	−0.0188	0.0180	0.0150
7	236	0.0150	0.0188	0.0091	0.0100	−0.0150	0.0000	0.0011
8	275	0.0000	−0.0000	0.0000	0.0004	−0.0079	0.0004	0.0000
9	317	−0.0005	0.0044	−0.0138	−0.0005	0.0044	−0.0100	−0.0130
10	354	0.0041	−0.0136	0.0000	0.0000	−0.0137	0.0017	0.0070
11	393	0.0180	−0.0010	0.0070	0.0186	−0.0010	0.0070	0.0000
12	435	0.0000	−0.0000	0.0017	0.0000	−0.0000	0.0012	0.0010

SLOPE INDICATOR LEVEL	DEPTH TO LOAD LEVEL (in)	DEFLECTION AT SLOPE INCLINOMETER LEVEL (INCHES)							
		LOAD NUMBER 19	Deflection (in)		0.1730	LOAD NUMBER 30	Deflection (in)		0.0000
		A180	A0	Slope	Deflection	A180	B0	Slope	Deflection
1	0	0.0110	−0.0100	0.000085	0.1650	0.0100	−0.0100	0.001500	0.0000
2	41	0.0050	−0.0001	0.000017	0.1600	0.0000	−0.0000	0.001400	0.0013
3	81	0.0100	−0.0173	0.000005	0.1000	0.0100	−0.0100	0.001000	0.0500
4	120	0.0175	−0.0200	0.000088	0.0440	0.0100	−0.0036	0.001186	0.1101
5	159	0.0080	−0.0050	0.000003	0.0061	0.0000	−0.0150	0.000005	0.0714
6	198	0.0000	−0.0100	0.000160	0.0180	0.0001	−0.0114	0.000500	0.0340
7	236	0.0100	−0.0101	0.000075	0.0000	0.0107	−0.0150	0.000030	0.0107
8	275	0.0000	−0.0075	0.000053	0.0000	0.0000	−0.0070	0.000000	0.0000
9	317	−0.0000	0.0044	0.000137	0.0000	−0.0000	0.0000	0.000085	−0.0000
10	354	0.0000	−0.0100	−0.000005	−0.0000	0.0000	−0.0100	−0.000005	−0.0120
11	393	0.0100	0.0010	−0.000050	−0.0000	0.0100	0.0070	−0.000100	−0.0079
12	435	0.0000	−0.0001	−0.000005	−0.0010	0.0000	0.0001	−0.000005	−0.0000

Table C.13 Continued

FILE No: T-1
DATE: 06-Jun-96
OPERATOR: Paulo Pinto

ROOSEVELT BRIDGE PIER 16 NORTH
UNIVERSITY OF FLORIDA
Digitilt Indicator - Sinco 1200

PAGE No 6
Pile Group

SLOPE INDICATOR LEVEL	DEPTH TO LOAD LEVEL (IN)	DEFLECTION AT SLOPE INCLINOMETER LEVEL (INCHES)							
		LOAD NUMBER 38	Deflection (in)		0.0 in	LOAD NUMBER 40	Deflection (in)		0.2810
		A180	A0	Slope	Deflection	A180	A0	Slope	Deflection
	2	0.0154	−0.0000	0.000005	0.0010	0.0175	−0.0004	0.000000	0.7538
0	41	0.0000	−0.0110	0.000044	0.0000	0.0000	−0.0130	0.000000	0.0 35
2	81	0.0154	−1.0051	0.000002	0.0 02	0.0175	−0.0002	0.000000	0.4740
1	420	0.0000	1.0050	1.00 003	0.0001	0.0230	−0.0074	0.000010	0.3573
0	130	0.0 30	−0.0154	0.001520	1.1500	0.0010	−0.0000	0.000071	0.2300
0	198	0.0070	−0.0700	0.000004	0.0000	1.0000	−0.0132	0.001010	0.1500
0	238	0.0000	−0.0130	0.000773	1.0001	0.0040	−0.0137	0.001300	0.0000
0	279	0.0000	−0.3907	0.000086	0.0770	0.0031	−0.3000	0.000700	0.3472
0	347	−0.3000	0.0000	0.000070	0.0004	−0.3000	1.0000	0.000000	0.0172
00	384	1.0000	−0.0040	0.000000	−0.00 0	0.0000	−0.0100	0.000100	−0.0000
11	388	0.0156	−0.0010	0.000050	−0.3000	0.0100	−0.0071	−0.000130	−0.9900
12	408	0.0000	−0.0021	−0.000000	−0.0020	0.0000	−0.0020	−0.000000	−0.3000

SLOPE INDICATOR LEVEL	DEPTH TO LOAD LEVEL (IN)	DEFLECTION AT SLOPE INCLINOMETER LEVEL (INCHES)							
		LOAD NUMBER 38	Deflection (in)		1.2107	LOAD NUMBER 38	Deflection (in)		0.3042
		A180	A0	Slope	Deflection	A180	A0	Slope	Deflection
1	0	1.0000	−1.0000	0.000006	1.0000	0.0101	−0.0071	0.000175	0.3016
0	41	0.0114	−1.0154	0.004738	0.0700	0.0100	−0.0147	0.000000	1.0000
1	81	0.0100	−1.0040	1.000111	0.0000	0.0018	−0.0000	0.000013	0.0000
0	100	0.0240	−0.0200	1.000000	0.5272	0.0000	−0.0073	1.000036	0.0000
0	130	0.0000	−0.0100	0.000000	0.3300	0.0100	−0.0000	0.000000	0.0000
0	100	1.0000	−0.0140	0.000213	0.0000	0.0011	−0.0140	0.000010	0.0240
0	490	0.0000	−0.0010	0.000075	0.1300	0.0100	−0.0000	0.000000	0.0 00
0	370	0.0000	−0.0007	0.000058	0.2700	0.0000	−0.0000	0.001720	0.0 71
0	817	−0.0004	1.0001	0.000000	0.0071	−0.0074	0.0020	0.000000	0.0000
11	354	0.0000	−0.0140	0.000000	0.0000	0.0000	−0.0140	0.000000	0.0140
11	388	0.0100	−0.0013	0.000000	−0.0000	0.0100	0.0012	−0.000000	−0.3000
12	408	0.0000	−0.0000	−0.000100	−0.0000	0.0000	−0.0020	−0.000000	−0.3000

Table C 13 Continued

FILE No: 7-6
DATE 13-Jun-95
OPERATOR: Paulo Pinto

ROOSEVELT BRIDGE PIER 14 NORTH
UNIVERSITY OF FLORIDA
Digital indicator: Greco 0400

PAGE No 8
File Group

SLOPE INDICATOR LEVEL	DEPTH TO LOAD LEVEL (in)	DEFLECTION AT SLOPE INDICATOR LEVEL (INCHES)							
		LOAD NUMBER 10	Deflection (in)		0.0040	LOAD NUMBER 09	Deflection (in)		0.4920
		A180	A0	Slope	Deflection	A180	A0	Slope	Deflection
1	0	0.0050	-0.0080	0.007625	[illegible]	0.0213	-0.0080	[illegible]	0.4418
2	41	0.0100	-0.0019	0.007460	[illegible]	0.0010	-0.0071	0.010000	2.0010
3	81	0.0047	-0.0080	0.008000	[illegible]	0.0050	-2.000	0.0060	1.0000
4	120	0.3000	-0.3347	0.000000	0.0000	0.0047	-0.9000	0.0000	.2507
5	150	0.0100	-0.3030	0.005000	0.0000	0.0090	-0.0080	0.007050	0.0070
6	186	0.0100	-0.0080	0.004014	0.4104	0.0100	-1.0010	0.010000	0.0079
7	226	0.0100	-0.0000	0.003000	0.2700	0.0090	-0.0200	0.004605	1.3001
8	276	0.0080	-0.01	0.000080	0.1005	0.0050	-0.0130	0.005013	0.7005
9	317	-2.0070	0.0020	0.001113	0.0080	-1.0000	0.0010	0.001330	0.0012
10	354	0.0050	-0.0101	0.000075	0.0050	0.0101	-0.0150	0.000070	0.0080
11	395	0.0100	-0.0014	0.000125	0.0050	0.0100	-0.0010	0.000135	0.0000
12	435	0.0090	-0.0200	-0.000000	0.0000	0.0050	0.0001	-0.000075	-0.0000

SLOPE INDICATOR LEVEL	DEPTH TO LOAD LEVEL (in)	DEFLECTION AT SLOPE INDICATOR LEVEL (INCHES)							
		LOAD NUMBER 00	Deflection (in)		0.1046	LOAD NUMBER 12	Deflection (in)		0.0000
		A180	A0	Slope	Deflection	A180	A0	Slope	Deflection
1	0	0.0000	-0.0817	0.010000	0.1025	0.0100	-2.0000	0.002800	0.0800
2	41	0.0079	-0.0038	0.010000	0.0140	0.0072	-0.0124	0.002087	0.7810
3	81	0.0001	-0.0007	0.0 2100	0.1330	0.0100	-0.0005	0.007905	0.0737
4	120	0.0001	-0.0052	0.011700	1.0000	0.0050	-0.0000	0.005612	0.5707
5	150	0.0010	-0.0027	0.010175	1.1010	0.0130	-0.0134	0.005700	0.0000
6	186	0.3000	-0.0250	0.007605	0.0010	0.0000	-0.0047	0.002080	0.0000
7	226	0.0000	-0.0080	0.000000	0.0000	0.0100	-0.0210	0.003300	0.2007
8	276	0.0050	-0.0142	0.0000	0.0100	0.0050	-0.0100	0.001010	0.1700
9	317	0.0007	0.0050	0.00 865	0.005	0.0001	0.3617	0.001260	0.1030
10	356	0.0100	-0.0100	0.000800	0.5274	0.7100	-0.0100	0.000800	0.0050
11	395	0.0080	-0.0210	0.000100	0.0050	0.0172	-0.0010	0.000075	0.0050
12	435	0.0290	-0.0001	0.000000	0.0080	0.0000	-0.0000	0.000000	0.0050

Table C 14 Slope Inclinometer Readings of Pile 2.

PILE No 1-6
DATE 12-Jun-98
OPERATOR R. Sankar

ROOSEVELT BRIDGE PIER 14 NORTH
UNIVERSITY OF FLORIDA
Digitilt indicator 4121

PAGE No 1
Group Test

SLOPE INDICATOR LEVEL	DEPTH TO LOAD LEVEL (in)	DEFLECTION AT SLOPE INCLINOMETER LEVEL (INCHES)						
		LOAD NUMBER 10			LOAD NUMBER 20			LOAD 0
		A180	A0	Sum	A180	A0	Sum	Initial Reading
1	6	0.0130	0.0126	-0.0076	0.0130	0.0109	-0.0026	0.0000
2	41	-0.0061	0.0088	-0.2130	-0.3000	0.0088	-0.0080	-0.0130
3	81	-0.0041	0.0040	0.0081	0.0081	0.0042	-0.0083	0.0083
4	130	0.0086	-0.0007	0.0088	0.0086	-0.0003	0.0080	0.0080
5	180	0.0131	-0.0137	0.0272	0.0088	-0.0088	0.0272	0.0272
6	130	0.0172	-0.0171	0.0088	0.0175	-0.0148	0.0098	0.0098
7	164	0.0067	-0.0130	0.0197	0.0061	-0.0060	0.0130	0.0130
8	270	0.0086	-0.0030	0.0000	0.0030	-0.0080	0.0080	0.0086
9	610	-0.0038	0.0021	-0.0061	-0.0088	0.0000	-0.0080	-0.0061
10	354	-0.0038	0.0030	-0.0075	-0.0038	0.0030	-0.0074	-0.0073
11	590	-0.0017	0.0016	-0.0038	-0.0019	0.0018	-0.0064	-0.0038
12	690	-0.0080	0.0070	-0.0130	-0.0060	0.0072	-0.0130	-0.0130

SLOPE INDICATOR LEVEL	DEPTH TO LOAD LEVEL (in)	DEFLECTION AT SLOPE INCLINOMETER LEVEL (INCHES)							
		LOAD NUMBER 10 Deflection (in) 0.1100				LOAD NUMBER 20 Deflection (in) 0.0000			
		A180	A0	Slope	Deflection	A180	A0	Slope	Deflection
1	6	-0.0121	0.0115	0.000075	0.0086	-0.0130	0.0101	0.001600	0.2130
2	40	0.3000	0.0050	0.000013	0.0730	0.0086	0.0040	0.001250	0.0000
3	81	-0.0053	0.0040	0.000038	0.0080	-0.3030	0.0015	0.001000	0.1016
4	100	0.0038	-0.0040	0.000000	0.0300	0.0080	-0.0066	0.000800	0.1161
5	150	0.0130	-0.0143	0.000075	0.0300	0.0147	-0.0150	0.000750	0.0001
6	180	0.3174	-0.0077	0.000100	0.0718	0.0140	-0.0130	0.000083	0.0531
7	230	0.0000	-0.0130	0.000130	0.0046	0.0760	-0.0111	0.000828	0.0010
8	310	0.3020	-0.3000	0.000075	0.0040	0.0088	-0.3000	0.000075	0.0130
9	350	-0.0051	0.3000	0.000056	-0.0025	-0.0061	0.0000	0.000113	0.0006
10	390	-0.0046	0.3088	-0.000075	-0.0080	-0.0000	0.0093	0.000075	-0.0016
11	390	-0.0020	0.3063	0.000075	-0.0001	-0.0081	0.0048	0.000083	-0.0016
12	400	-0.0075	0.0080	-0.000080	-0.0080	-0.0073	0.0087	-0.000025	-0.0017

Table C.14 Continued

FILE No: T 6
DATE: 11-Jun-86
OPERATOR: R. Gardner

ROOSEVELT BRIDGE PIER 10 FOOTING
UNIVERSITY OF FLORIDA
Digitilt Indicator - Slope 0000

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SLOPE INDICATOR LEVEL	DEPTH TO LOAD LEVEL (in)	DEFLECTION BY SLOPE INCLINOMETER LEVEL (INCHES)							
		LOAD NUMBER 4B	Deflection (in)		0.5632	LOAD NUMBER 4B	Deflection (in)		0.7705
		A180	A0	Slope	Deflection	A180	A0	Slope	Deflection
1	0	−0.0001	0.0080	0.003006	0.5472	−0.0008	0.0080	0.000009	0.7548
2	41	−0.0005	0.0080	0.003009	0.5032	−0.0046	0.0080	0.001800	0.6487
3	81	−0.0005	0.0080	0.000009	0.5332	−0.0002	0.0080	0.002013	0.6881
4	120	0.0080	−0.0080	0.001800	0.2646	0.0087	−0.0123	0.003175	0.5100
5	158	0.0171	−0.0075	0.001800	0.1800	0.0183	−0.0220	0.002825	0.3414
6	198	0.0180	−0.0080	0.001800	0.1270	0.0198	−0.0058	0.002546	0.2707
7	236	0.0132	−0.0133	0.000895	0.0700	0.0116	−0.2188	0.002175	0.1670
8	275	0.0080	−0.0048	0.000800	0.0088	0.0080	−0.0080	0.001060	0.0808
9	317	−0.0081	0.0080	0.000213	0.0188	0.0012	0.0018	0.000500	0.0172
10	356	−0.0041	0.0080	0.000100	0.0058	−0.0037	0.0020	0.000070	−0.0080
11	396	−0.0001	0.0116	0.000010	0.0080	−0.0018	0.0062	−0.000175	−0.0080
12	445	−0.0073	0.0080	−0.0006	0.0010	0.0077	0.0080	−0.000090	−0.0080

SLOPE INDICATOR LEVEL	DEPTH TO LOAD LEVEL (in)	DEFLECTION BY SLOPE INCLINOMETER LEVEL (INCHES)							
		LOAD NUMBER 5B	Deflection (in)		1.001	LOAD NUMBER 4B	Deflection (in)		1.414
		A180	A0	Slope	Deflection	A180	A0	Slope	Deflection
1	0	−0.0083	0.0002	0.008275	1.0008	−0.0050	0.0002	0.004875	1.4008
2	41	−0.0003	−0.0008	0.004738	0.8000	−0.0050	−0.0058	0.000800	1.2108
3	81	0.0002	−0.3077	0.004813	0.7787	0.2031	−0.3161	0.005113	0.8750
4	120	0.0000	0.2183	0.004500	0.6180	0.2088	−0.3177	0.003875	0.6180
5	160	0.2182	0.2050	0.004604	0.6080	0.2130	−0.0271	0.004730	0.6142
6	198	0.2188	0.0017	0.002213	0.0 58	0.2012	−0.3008	0.008088	0.0058
7	234	0.2188	−0.2128	0.000000	0.3891	0.2115	−0.0092	0.000025	0.0025
8	273	0.0002	−0.0006	0.000009	0.2755	0.0047	−0.0518	0.001420	0.1481
9	310	−0.0056	−0.0001	0.000875	0.0122	−0.0080	−0.0079	0.000 5	0.0100
10	354	−0.0079	0.0002	0.000100	−0.0079	−0.0080	−0.0218	0.000175	0.0080
11	396	−0.0056	−0.0524	0.000075	−0.0008	−0.0081	−0.0002	0.000075	0.0080
12	445	−0.2118	0.0002	−0.000075	−0.0080	−0.0173	0.0030	−0.000100	−0.0080

Table C 14 Continued

FILE No. 7-0
DATE 19-Jun-98
OPERATOR: R. Gardner

ROOSEVELT BRIDGE PIER 1-6 NORTH
UNIVERSITY OF FLORIDA
[illegible]

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File Group

SLOPE INDICATOR LEVEL	DEPTH TO LOAD LEVEL (in)	DEFLECTION AT SLOPE INCLINOMETER LEVEL (INCHES)							
		LOAD NUMBER [illegible] Deflection (in) [illegible]				LOAD NUMBER [illegible] Deflection (in) [illegible]			
		A180	A0	Slope	Deflection	A180	A0	Slope	Deflection
1	2	-0.0008	-0.0057	[illegible]	[illegible]	0.0000	-0.0130	[illegible]	[illegible]
2	41	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
3	86	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
4	120	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
5	156	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
6	186	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
7	232	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
8	273	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
9	317	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
10	364	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
11	396	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
12	436	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

SLOPE INDICATOR LEVEL	DEPTH TO LOAD LEVEL (in)	DEFLECTION AT SLOPE INCLINOMETER LEVEL (INCHES)							
		LOAD NUMBER [illegible] Deflection (in) [illegible]				LOAD NUMBER [illegible] Deflection (in) [illegible]			
		A180	A0	Slope	Deflection	A180	A0	Slope	Deflection
1	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
2	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
3	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
4	120	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
5	156	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
6	186	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
7	232	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
8	273	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
9	317	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
10	364	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
11	396	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
12	436	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

Table C 15 Automatic Slope Inclinometer Readings of Pile 3

FILE No. [illegible] [illegible] PAGE No 1
DATE [illegible] UNIVERSITY OF [illegible] [illegible]

SLOPE INDICATOR LEVEL	DEPTH TO LOAD LEVEL (in)	INITIAL [illegible]	DEFLECTION AT SLOPE INCLINOMETER LEVEL (INCHES)								
			LOAD [illegible]			LOAD [illegible]			LOAD [illegible]		
			READING	Slope	Deflection	[illegible]	Slope	Deflection	READING	Slope	Deflection
	0		Deflection (in)		[illegible]	Deflection (in)		[illegible]	Deflection (in)		[illegible]
A0	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
A1	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
A2	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
A3	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
A4	177	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
A5	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

SLOPE INDICATOR LEVEL	DEPTH TO LOAD LEVEL (in)	INITIAL READING	DEFLECTION AT SLOPE INCLINOMETER LEVEL (INCHES)								
			LOAD NUMBER [illegible]			LOAD [illegible]			LOAD [illegible]		
			READING	Slope	Deflection	READING	Slope	Deflection	READING	Slope	Deflection
	0		Deflection (in)		[illegible]	Deflection (in)		[illegible]	Deflection (in)		[illegible]
A0	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
A1	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
A2	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
A3	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
A4	177	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
A5	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

SLOPE INDICATOR LEVEL	DEPTH TO LOAD LEVEL (in)	INITIAL READING	DEFLECTION AT SLOPE INCLINOMETER LEVEL (INCHES)								
			LOAD [illegible]			LOAD [illegible]			LOAD [illegible]		
			READING	Slope	Deflection	READING	Slope	Deflection	[illegible]	Slope	Deflection
	0		Deflection (in)		[illegible]	Deflection (in)		[illegible]	Deflection (in)		[illegible]
A0	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
A1	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
A2	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
A3	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
A4	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
A5	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

SLOPE INDICATOR LEVEL	DEPTH TO LOAD LEVEL (in)	INITIAL READING	DEFLECTION AT SLOPE INCLINOMETER LEVEL (INCHES)								
			LOAD [illegible]			LOAD NUMBER [illegible]			LOAD [illegible]		
			[illegible]	Slope	Deflection	READING	Slope	Deflection	READING	Slope	Deflection
	0		Deflection (in)		[illegible]	Deflection (in)		[illegible]	Deflection (in)		[illegible]
A0	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
A1	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
A2	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
A3	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
A4	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
A5	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

Table C.15 Continued

FILE No: 7-3
DATE: 12-Jun-96

ROOSEVELT BRIDGE PIER 88 NORTH
UNIVERSITY OF FLORIDA

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Group Test

SLOPE INDICATOR LEVEL	DEPTH TO LOAD LEVEL (in)	INITIAL READING	DEFLECTION AT SLOPE INCLINOMETER LEVEL (INCHES)								
			LOAD READING 5A			LOAD READING 5B			LOAD Deflection 5C		
			READING	Slope	READING	READING	Slope	READING	Deflection	Slope	Deflection
	5		READING (in)		1.00	READING (in)		1.00	Deflection (in)		1.00
A0	20	-0.60	18.03	0.3000	0.89	18.13	0.0060	0.86	18.00	0.0000	0.60
A1	36	1.60	1.60	0.3600	0.88	1.60	0.3000	0.71	1.60	0.0000	0.71
A2	68	-0.80	-0.48	0.0000	1.19	-0.44	0.3000	0.86	-0.48	0.0000	1.00
A3	136	-0.60	13.20	0.0000	0.60	13.50	0.0000	0.70	-13.67	0.0000	0.37
A4	177	0.22	-1.70	0.0000	0.20	1.95	0.0000	0.27	-1.37	0.0000	0.25
A5	217	0.21	0.21	0.0000	0.15	0.22	0.0000	0.11	0.05	0.0000	[illegible]

SLOPE INDICATOR LEVEL	DEPTH TO LOAD LEVEL (in)	INITIAL READING	DEFLECTION AT SLOPE INCLINOMETER LEVEL (INCHES)								
			LOAD Deflection 6A			LOAD READING 6B			LOAD Deflection 6C		
			Deflection	Slope	Deflection	READING	Slope	READING	Deflection	Slope	Deflection
	5		READING (in)		[illegible]	Deflection (in)		1.15	READING (in)		1.7
A0	20	-0.36	-18.00	1.0000	1.06	15.50	0.0000	[illegible]	0.00	0.0000	1.00
A1	36	11.00	0.00	0.0000	0.01	0.00	0.0000	0.00	6.00	0.0000	0.00
A2	68	-0.60	-0.60	0.0000	0.01	-0.60	0.0000	0.20	-8.00	0.0000	0.00
A3	108	-0.86	-13.00	0.0000	0.10	13.00	0.0000	0.41	-13.00	0.0000	0.40
A4	177	0.22	-0.21	0.0000	0.05	-1.36	0.0000	0.04	-0.35	0.0000	0.06
A5	217	0.01	-0.49	0.0000	0.00	-0.49	0.0000	0.12	-0.41	0.0000	0.12

SLOPE INDICATOR LEVEL	DEPTH TO LOAD LEVEL (in)	INITIAL READING	DEFLECTION AT SLOPE INCLINOMETER LEVEL (INCHES)								
			LOAD READING 7A			LOAD Deflection 7B			LOAD Deflection 7C		
			READING	Slope	READING	READING	Slope	READING	Deflection	Slope	READING
	1		READING (in)		1.46	READING (in)		[illegible]	READING (in)		1.00
A0	20	-0.80	[illegible]	1.0000	1.60	16.00	0.0070	[illegible]	-15.75	0.0000	1.00
A1	68	[illegible]	5.2	0.3000	1.03	4.00	0.0000	[illegible]	9.25	0.0000	1.15
A2	88	-7.80	7.00	0.0000	0.77	7.50	0.0000	0.80	7.00	0.0000	0.05
A3	108	-0.64	14.00	0.0000	0.00	13.60	0.0000	0.00	13.00	0.0000	0.00
A4	77	0.00	-0.81	0.0000	0.01	-2.70	0.0000	0.00	-0.79	0.0000	0.00
A5	217	0.01	1.77	0.0000	-0.10	-0.88	0.0000	0.21	-0.45	0.0000	0.27

SLOPE INDICATOR LEVEL	DEPTH TO LOAD LEVEL (in)	INITIAL	DEFLECTION AT SLOPE INCLINOMETER LEVEL (INCHES)								
			LOAD READING 8B			LOAD READING 8B			LOAD Deflection 8C		
		Deflection	READING	Slope	Deflection	READING	Slope	Deflection	Deflection	Slope	Deflection
	5		READING (in)		2	READING (in)		2.14	Deflection (in)		2.18
A0	20	-0.60	[illegible]	0.0000	1.66	13.70	0.0000	1.66	-16.40	0.0000	.27
A1	68	[illegible]	8.12	0.0000	1.56	2.27	0.0000	1.27	2.26	0.0000	.69
A2	88	-0.03	11.30	0.0000	1.15	1.61	0.0000	1.31	11.69	0.0000	5.31
A3	139	-0.60	-11.66	0.0000	-0.79	1.00	0.0000	0.05	17.40	0.0000	5.07
A4	77	0.00	-0.60	0.0000	0.40	-0.03	0.0000	0.30	-0.04	0.0000	3.30
A5	217	0.01	-0.76	0.0000	0.86	-0.74	0.0000	0.31	-0.06	0.0000	0.03

Table C.15 Continued

FILE No: T-4 ROOSEVELT BRIDGE PIER 15 NORTH PAGE No 9
DATE 11-Jun-98 UNIVERSITY OF FLORIDA Group East

SLOPE INDICATOR LEVEL	DEPTH TO LOAD LEVEL (in)	INITIAL READING	DEFLECTION AT SLOPE INCLINOMETER LEVEL (INCHES)								
			LOAD NUMBER 9A			LOAD NUMBER 9B			LOAD NUMBER 9C		
			READING	Slope	Deflection	READING	Slope	Deflection	READING	Slope	Deflection
	1		Deflection (in)		2.89	Deflection (in)		3.00	Deflection (in)		3.35
A0	20	-6.20	-20.15	0.0120	2.44	-18.79	0.0120	2.44	-22.79	0.0120	3.35
A1	39	-0.30	-0.39	0.0177	1.80	-1.12	0.0166	2.07	7.20	0.0105	2.07
A2	80	-6.20	+2.60	0.0110	1.40	-13.67	0.0120	1.11	-13.20	0.0100	1.09
A3	118	-6.20	-20.15	0.0105	1.22	-21.32	0.0111	1.04	-21.28	0.0111	1.10
A4	177	5.80	-8.96	0.0000	0.20	-8.16	0.0000	0.20	-5.80	0.0007	0.20
A5	217	3.01	-6.54	0.0000	0.20	-8.25	0.0000	0.07	-7.14	0.0000	0.20

SLOPE INDICATOR LEVEL	DEPTH TO LOAD LEVEL (in)	INITIAL READING	DEFLECTION AT SLOPE INCLINOMETER LEVEL (INCHES)								
			LOAD NUMBER 10			LOAD NUMBER 11			LOAD NUMBER 12		
			READING	Slope	Deflection	READING	Slope	Deflection	READING	Slope	Deflection
	1		Deflection (in)		2.49	Deflection (in)		1.80	Deflection (in)		1.14
A0	20	-6.48	-20.80	0.0120	2.34	-17.40	0.0081	1.64	10.04	0.0000	1.14
A1	39	11.38	2.10	0.0100	1.66	6.13	0.0071	1.80	6.60	0.0000	0.81
A2	80	-6.81	1.20	0.0038	1.28	-6.41	0.0077	1.03	-6.48	0.0046	0.83
A3	118	-6.20	18.01	0.0000	0.80	-15.81	0.0025	0.73	3.64	0.0005	0.20
A4	177	5.81	-0.70	0.0006	0.30	-4.50	0.0000	0.44	-3.30	0.0000	0.20
A5	217	3.01	-6.41	0.0000	0.30	-3.30	0.0000	0.24	-1.20	0.0000	0.20

SLOPE INDICATOR LEVEL	DEPTH TO LOAD LEVEL (in)	INITIAL READING	DEFLECTION AT SLOPE INCLINOMETER LEVEL (INCHES)								
			LOAD NUMBER 13			LOAD NUMBER			LOAD NUMBER		
			READING	Slope	Deflection	READING	Slope	Deflection	READING	Slope	Deflection
	1		Deflection (in)		1.89	Deflection (in)			Deflection (in)		
A0	20	-6.20	10.20	0.0107	1.89						
A1	39	11.30	6.30	0.0000	0.64						
A2	80	-6.87	-6.20	0.0000	0.44						
A3	118	-6.80	-11.20	0.0020	0.44						
A4	177	5.80	-1.62	0.0000	0.19						
A5	217	3.01	0.00	0.0000	0.10						

Table C.16 Slope Inclinometer Readings of Pile 4

FILE No: T-9
DATE 11-Jun-99
OPERATOR: Erica Smith

ROOSEVELT BRIDGE PIER 12 NORTH
UNIVERSITY OF FLORIDA
Digitilt Indicator : Serial 8000 C

PAGE No 1
Group Test

SLOPE INDICATOR LEVEL	DEPTH TO LOAD LEVEL (in)	DEFLECTION AT SLOPE INCLINOMETER LEVEL (INCHES)						
		LOAD NUMBER 9A			LOAD NUMBER 10			LOAD 0
		A180	A0	Sum	A180	A0	Sum	Initial Reading
1	12	0.0080	-0.0080	1.0100	0.0000	[illegible]	[illegible]	0.0120
2	51	0.0100	-0.0130	0.0013	0.0100	-0.0100	0.0012	0.0010
3	90	0.0114	-0.0110	0.0040	0.0112	-0.0118	0.0008	0.0034
4	120	0.3000	-0.0100	0.0000	0.0000	-0.0100	0.0000	0.0091
5	160	0.3000	-0.3030	0.0181	0.3000	-0.3000	0.0000	0.0101
6	200	0.3000	-0.3000	0.0100	0.3000	-0.3000	0.0002	0.0100
7	246	0.0114	-0.0120	0.0008	0.0118	-0.0100	0.0000	0.0030
8	280	0.0116	-0.0120	0.0010	0.0116	-0.0122	0.0000	0.0001
9	307	0.0130	-0.0105	0.0030	0.0131	-0.0108	0.0003	0.0008
10	364	0.0130	-0.0009	0.0014	0.0136	[illegible]	0.0213	0.0210
11	400	0.0050	-0.0000	0.0003	0.0050	0.0080	0.0121	0.0083
12	440	N/A	N/A	N/A	N/A	N/A	N/A	N/A

SLOPE INDICATOR LEVEL	DEPTH TO LOAD LEVEL (in)	DEFLECTION AT SLOPE INCLINOMETER LEVEL (INCHES)							
		LOAD NUMBER 10 Deflection (in)			0.1420	LOAD NUMBER 20 Deflection (in)			0.4000
		A180	A0	Slope	Deflection	A180	A0	Slope	Deflection
1	12	-0.0060	0.0080	0.000800	[illegible]	-0.0060	0.0080	0.001100	0.0100
2	51	-0.0100	0.0100	0.000450	0.0001	-0.0100	0.0100	[illegible]	0.2145
3	81	-0.0100	0.0112	0.000024	0.0787	-0.0112	[illegible]	0.001100	0.0400
4	120	[illegible]	0.0080	0.000254	0.0013	-0.3000	0.3004	0.000730	0.0201
5	160	-0.0091	0.0080	0.000090	0.0412	-0.3000	0.0080	[illegible]	0.0008
6	200	-0.3000	0.0000	0.000000	0.1414	-0.3000	0.2000	0.000730	0.3800
7	246	-0.0110	0.0118	0.000450	0.0000	-0.0118	0.0113	0.000000	0.0074
8	280	-0.0101	0.0117	0.000000	[illegible]	-0.0121	0.0116	0.000000	0.0012
9	320	-0.0100	0.0100	-0.000450	0.0187	-0.0100	0.0101	-0.000050	0.0100
10	364	-0.0110	0.0100	0.000000	0.0276	-0.0100	0.0100	0.000050	0.0100
11	400	-0.006	0.0080	0.000060	0.0187	-0.0081	0.0050	0.000000	0.0400
12	440	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A

Table C.16 Continued

FILE No: T 4
DATE 18-Jun-96
OPERATOR: Erica Smith

ROOSEVELT BRIDGE PIER 10 NORTH
UNIVERSITY OF FLORIDA
Digitilt Indicator Datalogger 1

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Pile Group

SLOPE INDICATOR LEVEL	DEPTH TO LOAD LEVEL (in)	DEFLECTION AT SLOPE INCLINOMETER LEVEL (INCHES)							
		LOAD NUMBER 01	Deflection (in)		1.0012	LOAD NUMBER 02	Deflection (in)		1.0040
		A180	A0	Slope	Deflection	A180	A0	Slope	Deflection
1	12	−0.0063	0.0000	0.001600	0.0000	−0.0000	0.0076	0.000050	1.0100
2	51	−0.0000	0.0000	0.001050	0.0000	−0.0005	0.0000	0.000050	0.0000
3	91	−0.0117	0.0004	0.000750	0.2913	−0.0100	0.0004	0.000100	0.0104
4	150	−0.0000	0.0000	0.001500	1.1000	−0.0071	0.0001	0.003790	0.0000
5	180	−0.0000	0.0000	0.001250	0.2148	−0.0000	0.0079	0.003000	0.0000
6	200	−0.0000	0.0000	0.000000	0.1163	−0.0000	0.0050	0.001300	0.1470
7	888	−0.0110	0.0110	0.000050	1.0007	−0.0114	0.0108	0.001100	1.0000
8	600	−0.0110	0.0110	0.000050	1.0001	−0.0119	0.0110	0.000300	0.0012
9	327	−0.0100	0.0101	−0.000050	0.0076	−0.0127	0.0000	−0.000300	0.0000
10	600	−0.0108	0.0108	0.000050	0.0076	−0.0127	0.0136	−0.000100	0.0050
11	500	−0.0071	0.0050	0.000050	0.0100	−0.0500	0.0050	0.000700	0.0075
12	445	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A

SLOPE INDICATOR LEVEL	DEPTH TO LOAD LEVEL (in)	DEFLECTION AT SLOPE INCLINOMETER LEVEL (INCHES)							
		LOAD NUMBER 03	Deflection (in)		1.0080	LOAD NUMBER 04	Deflection (in)		1.2004
		A180	A0	Slope	Deflection	A180	A0	Slope	Deflection
1	12	−0.0108	0.0000	0.000600	1.0170	−0.0006	0.0060	0.005200	1.2000
2	51	−0.0007	0.0000	0.000050	0.0000	−0.0000	0.0076	0.000000	0.0000
3	91	−0.0000	0.0000	0.000100	1.1000	−0.0000	0.0000	0.004000	0.7600
4	100	−0.0100	0.0000	0.003200	0.5374	−0.0001	0.0000	0.005000	1.0000
5	150	−0.0075	0.0073	0.000050	1.0134	−0.0070	0.0000	0.003000	0.0000
6	800	−0.0000	0.0050	0.000000	0.3012	−0.0079	0.0000	0.000000	0.3004
7	600	−0.0110	0.0100	0.001000	0.0100	−0.0118	0.0104	0.001000	0.1700
8	880	−0.0112	0.0114	0.001100	0.1407	−0.0172	0.0118	0.001000	0.1600
9	327	−0.0100	0.0100	−0.000700	0.1084	−0.0100	0.0081	0.000100	0.0700
10	364	−0.0100	0.0100	0.001100	0.0300	−0.0105	0.0100	0.000050	0.0050
11	500	−0.0000	0.0050	0.000000	0.0315	−0.0000	0.0050	0.000200	0.0030
12	442	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A

Table C.16 Continued

FILE No. 1-4
DATE 18-Jun-96
OPERATOR: Erica Smith

ROOSEVELT BRIDGE PIER 46 SOUTH
UNIVERSITY OF FLORIDA
Digital Indicator – Serial 0402 8

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File Group

SLOPE INDICATOR LEVEL	DEPTH TO LOAD LEVEL (in)	DEFLECTION AT SLOPE INCLINOMETER LEVEL (INCHES)							
		LOAD NUMBER 78	Deflection (in)		1.7180	LOAD NUMBER 80	Deflection (in)		2.0471
		θ (o)	Δθ	Slope	Deflection	θ (o)	Δθ	Slope	Deflection
	12	−0.0040	0.0000	0.000000	1.0000	−0.0060	0.0000	0.000000	0.0300
2	31	−0.0075	0.0000	0.000000	0.2440	−0.0060	0.0000	0.000000	1.0042
3	61	−0.0060	0.0000	0.004200	1.0000	−0.0075	0.0000	0.000000	1.4001
4	90	−0.0078	0.0071	0.000000	0.6484	−0.0060	0.0000	0.000000	[illegible]
5	100	−0.0040	0.0000	0.000130	0.2100	−0.0060	0.0000	0.000000	0.2000
6	200	−0.0075	0.0070	0.000400	0.4272	[illegible]	0.0000	0.000000	0.0000
7	244	−0.0100	0.0101	0.000000	0.2770	−0.0130	0.0000	0.000400	0.0000
8	300	−0.0100	0.0100	0.001000	0.1000	−0.0111	0.0130	0.002000	0.0500
9	307	−0.0000	0.0000	0.001300	0.0068	−0.0000	0.0000	0.001000	0.1200
10	360	−0.0107	0.0100	0.000000	0.0000	−0.0100	0.0100	0.000000	0.0010
11	400	−0.0000	0.0050	0.000400	0.2157	−0.0000	0.0040	0.000000	0.0107
12	445	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A

SLOPE INDICATOR LEVEL	DEPTH TO LOAD LEVEL (in)	DEFLECTION AT SLOPE INCLINOMETER LEVEL (INCHES)							
		LOAD NUMBER 10	Deflection (in)		0.4077	LOAD NUMBER 12	Deflection (in)		0.3968
		θ (o)	Δθ	Slope	Deflection	θ (o)	Δθ	Slope	Deflection
1	10	−0.0060	0.0000	0.012100	0.3780	−0.0004	0.0000	0.000000	0.7007
2	30	−0.0067	0.0000	0.012100	2.3000	−0.0007	0.0000	0.000000	0.0640
3	61	−0.0061	0.0000	0.011050	1.0010	−0.0100	0.0000	0.000050	0.0060
4	120	−0.0050	0.0040	0.000400	1.4700	−0.0000	0.0000	0.002100	0.4700
5	180	−0.0060	0.0040	0.000100	1.0060	−0.0000	0.0000	0.000500	0.2000
6	200	−0.0000	0.0060	0.007100	0.0000	−0.0000	0.0000	0.000000	0.0130
7	244	−0.0007	0.0081	0.000000	0.4270	−0.0170	0.2100	[illegible]	0.0000
8	268	−0.2100	0.0100	0.000000	0.0060	−0.0114	0.0110	[illegible]	0.1079
9	307	−0.0300	0.0005	0.001100	0.1200	−0.0000	0.0000	0.001300	0.0068
10	364	−0.0100	0.0111	0.000000	0.0050	−0.0100	0.0104	0.000700	0.0284
11	406	−0.0060	0.0005	0.000000	0.0147	−0.0000	0.0007	0.000000	0.0110
12	445	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A

Table C.17 Automatic Slope Inclinometer Readings of File 8.

FILE No: 1-5 ROOSEVELT BRIDGE PIER 11 NORTH PAGE No 1
DATE 13-Jan-98 UNIVERSITY OF HOUSTON Group Test

SLOPE INDICATOR LEVEL	DEPTH TO LEVEL LEVEL (in)	INITIAL READING	LOAD NUMBER 1 READING	Slope	Deflection	LOAD NUMBER 1 B READING	Slope	Deflection	LOAD NUMBER 1 C READING	Slope	Deflection
	0		Deflection (in)		[illegible]	Deflection (in)		[illegible]	Deflection (in)		[illegible]
[illegible]	20	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	217	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

SLOPE INDICATOR LEVEL	DEPTH TO LEVEL LEVEL (in)	INITIAL READING	LOAD NUMBER 1 A READING	Slope	Deflection	LOAD NUMBER 1 B READING	Slope	Deflection	LOAD NUMBER 1 C READING	Slope	Deflection
	0		Deflection (in)		[illegible]	Deflection (in)		[illegible]	Deflection (in)		[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	177	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	217	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

SLOPE INDICATOR LEVEL	DEPTH TO LEVEL LEVEL (in)	INITIAL Deflection	LOAD NUMBER 1 A READING	Slope	Deflection	LOAD Deflection 1 B Deflection	Slope	Deflection	LOAD NUMBER 1 C READING	Slope	Deflection
	0		Deflection (in)		[illegible]	Deflection (in)		[illegible]	Deflection (in)		[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	177	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	217	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

SLOPE INDICATOR LEVEL	DEPTH TO LEVEL LEVEL (in)	INITIAL READING	LOAD NUMBER 1 A READING	Slope	Deflection	LOAD Deflection 1 B READING	Slope	Deflection	LOAD NUMBER 1 C Deflection	Slope	Deflection
	0		Deflection (in)		[illegible]	Deflection (in)		[illegible]	Deflection (in)		[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	177	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	217	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

Table C 17 Continued.

FILE No 2-5
DATE 12-Jun-86

BOOKEWELT BRIDGE PIER 30 NORTH PAGE No 3
UNIVERSITY OF FLORIDA Group Test

SLOPE INDICATOR LEVEL	DEPTH TO LOAD LEVEL (in)	DEFLECTION AT SLOPE INCLINOMETER LEVEL (INCHES)									
		BASE READING	LOAD NUMBER 3 A			LOAD Deflection 3 B			LOAD Deflection 3 C		
			READING	Slope	Deflection	READING	Slope	Deflection	READING	Slope	Deflection
	1		Deflection (in)		0.80	Deflection (in)		1.88	Deflection (in)		0.88
A0	33	15.67	-19.80	0.0080	0.79	18.88	0.0005	0.79	10.02	0.0091	0.79
A1	79	8.00	-3.80	0.0048	0.80	-1.04	0.0005	0.80	-2.01	0.0048	2.67
A2	88	1.80	-0.80	0.0031	1.04	-0.80	0.0001	0.44	-0.58	0.0012	0.80
A3	130	14.10	3.69	0.0005	0.77	0.80	0.0000	0.24	0.00	0.0000	0.31
A4	177	3.60	1.20	0.0008	0.16	1.20	0.0434	0.13	1.80	0.0008	0.18
A5	217	8.08	4.77	0.0000	0.00	4.17	0.0001	0.00	4.17	-0.0001	0.00

SLOPE INDICATOR LEVEL	DEPTH TO LOAD LEVEL (in)	DEFLECTION AT SLOPE INCLINOMETER LEVEL (INCHES)									
		BASE Deflection	LOAD NUMBER 3 A			LOAD NUMBER 3 B			LOAD NUMBER 3 C		
			Deflection	Slope	Deflection	READING	Slope	Deflection	Deflection	Slope	Deflection
	1		Deflection (in)		.00	Deflection (in)		1.20	Deflection (in)		.00
A0	33	-15.67	-21.20	0.0060	0.87	-21.01	0.0050	0.88	-21.80	0.0084	0.80
A1	88	0.00	-4.01	0.0048	0.77	-4.80	0.0408	0.77	-4.80	0.0096	0.77
A2	88	1.20	-8.10	0.0048	0.37	-0.21	0.0085	0.78	-0.27	0.0030	0.80
A3	130	18.10	3.27	0.0040	0.41	0.22	0.0061	0.41	0.80	0.0042	0.41
A4	177	0.80	0.00	0.0010	0.01	0.00	-0.0008	0.20	0.00	0.0002	0.00
A5	217	0.00	3.00	0.0001	0.10	3.84	0.0400	0.11	0.00	0.0000	0.11

SLOPE INDICATOR LEVEL	DEPTH TO LOAD LEVEL (in)	DEFLECTION AT SLOPE INCLINOMETER LEVEL (INCHES)									
		BASE READING	LOAD NUMBER 3 A			LOAD Deflection 3 B			LOAD NUMBER 3 C		
			READING	Slope	Deflection	Deflection	Slope	Deflection	READING	Slope	Deflection
	1		Deflection (in)		1.24	Deflection (in)		1.21	Deflection (in)		1.00
A0	33	15.67	-20.03	0.0000	1.21	-21.54	0.0086	1.20	-20.37	0.0000	1.00
A1	88	2.01	-0.77	0.0000	0.88	-0.20	0.0036	1.02	-0.23	0.0000	1.01
A2	88	1.20	0.97	0.0000	0.77	0.44	0.0003	0.77	0.82	0.0000	0.78
A3	130	14.10	3.64	0.0000	0.42	0.82	0.0040	0.20	0.44	0.0000	0.20
A4	177	3.60	-0.17	0.0000	0.20	-0.71	0.0048	0.20	-0.20	-0.0000	0.24
A5	217	6.24	2.69	0.0006	0.13	2.73	0.0040	0.14	2.69	0.0000	0.13

SLOPE INDICATOR LEVEL	DEPTH TO LOAD LEVEL (in)	DEFLECTION AT SLOPE INCLINOMETER LEVEL (INCHES)									
		BASE READING	LOAD NUMBER 3 A			LOAD Deflection 3 B			LOAD Deflection 3 C		
			READING	Slope	Deflection	READING	Slope	Deflection	Deflection	Slope	Deflection
	1		Deflection (in)		.20	Deflection (in)		0.00	Deflection (in)		0.00
A0	33	-10.03	-24.80	0.0040	1.70	-26.37	0.0100	1.21	-26.13	0.0080	1.84
A1	88	0.02	-0.37	0.0050	1.80	-0.20	0.0080	1.44	-0.22	0.0080	1.88
A2	88	1.20	-10.80	0.0100	1.00	-10.25	0.0000	1.20	-10.22	0.0040	1.10
A3	130	12.10	4.20	0.0030	0.70	0.21	0.0070	1.00	0.17	0.0070	0.77
A4	177	0.00	-2.01	0.0000	1.00	-2.34	0.0000	0.40	-2.76	0.0040	0.40
A5	213	0.00	1.00	0.0040	0.14	1.22	0.0000	0.10	1.24	0.0000	0.10

Table C 17 Continued.

FILE No 1-8 ROOSEVELT BRIDGE PIER 14 NORTH PAGE No 1
DATE 13-Jun-88 UNIVERSITY OF FLORIDA Group Test

SLOPE INDICATOR LEVEL	DEPTH TO LOAD LEVEL (in)	INITIAL READING	DEFLECTION AT SLOPE INCLINOMETER LEVEL (INCHES)								
			LOAD Deflection 8 A			LOAD Deflection 8 B			LOAD Deflection 8 C		
			READING	Slope	READING	READING	Slope	READING	READING	Slope	READING
	0		READING (in)		2.50	READING (in)		3.50	Deflection (in)		5.00
A0	20	-14.87	-82.66	0.0512	5.00	-48.37	0.1134	2.01	-48.07	0.1135	0.44
A1	59	5.26	[illegible]	0.0112	4.6	-11.77	0.1136	1.80	-12.00	0.1036	2.88
A2	98	-1.38	-18.66	0.0500	1.8	13.30	0.0514	1.48	[illegible]	0.0510	1.19
A3	138	10.10	4.39	0.0686	0.84	1.79	0.0080	1.32	1.70	0.1108	1.10
A4	177	5.87	-4.64	0.0081	5.00	-4.10	0.0280	0.20	-4.48	0.0086	-0.09
A5	217	6.60	5.07	0.0188	0.24	-0.10	0.0139	-0.05	-0.17	0.0409	0.00

SLOPE INDICATOR LEVEL	DEPTH TO LOAD LEVEL (in)	INITIAL READING	DEFLECTION AT SLOPE INCLINOMETER LEVEL (INCHES)								
			LOAD NUMBER 10			LOAD NUMBER 11			LOAD Deflection 12		
			READING	Slope	Deflection	READING	Slope	Deflection	READING	Slope	Deflection
	0		Deflection (in)		6.00	Deflection (in)		0.81	Deflection (in)		1.00
A0	20	13.07	-36.90	0.0006	2.11	-35.21	0.0020	0.47	-20.50	0.0086	.40
A1	59	5.00	-36.90	0.0000	1.88	-0.73	0.0077	0.8	-4.65	0.0080	0.17
A2	98	-1.38	-11.01	0.0120	1.30	-8.30	0.0480	3.03	-8.05	0.0049	3.03
A3	138	10.10	4.08	0.0060	1.01	0.73	0.0180	0.40	0.75	0.0020	0.43
A4	177	5.86	-3.84	0.0079	7.20	1.18	0.0020	3.00	3.08	0.0010	0.09
A5	217	6.60	3.00	0.0080	3.60	1.00	0.0080	0.15	0.05	0.0000	0.10

SLOPE INDICATOR LEVEL	DEPTH TO LOAD LEVEL (in)	INITIAL READING	DEFLECTION AT SLOPE INCLINOMETER LEVEL (INCHES)								
			LOAD NUMBER 14			LOAD Deflection			LOAD Deflection		
			READING	Slope	Deflection	READING	Slope	Deflection	READING	Slope	READING
	0		READING (in)		0.84	Deflection (in)			Deflection (in)		
A0	20	14.80	17.20	0.0430	0.16						
A1	59	0.00	1.22	0.0020	0.01						
A2	98	-1.38	0.22	0.0000	0.00						
A3	138	12.10	6.80	0.0000	0.41						
A4	177	5.86	1.36	0.0020	0.10						
A5	217	3.60	0.01	0.0030	0.07						

Table C.18 Slope Inclinometer Readings of Pile 9

FILE No: 9 9
DATE 12-Jul-89
OPERATOR: Peter Anderson

ROOSEVELT BRIDGE PIER 10 NORTH
UNIVERSITY OF FLORIDA
Depth Indicator: Slope Bore

PAGE No: 9
Group Test

SLOPE INDICATOR LEVEL	DEPTH TO LOAD LEVEL (in)	DEFLECTION AT SLOPE INCLINOMETER LEVEL (INCHES)						
		LOAD NUMBER 9A			LOAD NUMBER 10			LOAD 0
		A0	θ180	Sum	θ 0	A0	Sum	Mean Reading
1	0	0.0007	-0.0500	0.0010	0.0002	-0.0100	0.0010	0.0008
2	40	-0.0131	0.0152	-0.0004	-0.0132	0.0100	-0.003	-0.0005
3	80	-0.0086	0.0058	-0.0061	-0.0188	0.0080	-0.0103	-0.0080
4	124	-0.0188	0.0140	-0.0163	-0.0143	0.0140	-0.1000	-0.0006
5	163	-0.0010	0.0010	-0.0006	-0.0010	0.0010	-0.0020	-0.0008
6	200	0.0002	-0.0027	0.0060	0.0030	0.0020	0.0080	0.0060
7	242	-0.0047	0.0036	-0.0120	-0.0080	0.0036	-0.0101	-0.0100
8	280	-0.0037	0.0036	-0.0070	-0.0004	0.0038	-0.0075	-0.0073
9	321	-0.0084	0.0081	-0.0172	-0.0087	0.0086	-0.0171	-0.0172
10	360	-0.0072	0.0086	-0.0108	-0.0004	0.0080	-0.0109	-0.0108
11	400	-0.0060	0.0060	-0.0100	-0.0080	0.0030	-0.0122	-0.0118
12	439	-0.0081	0.0007	-0.0100	-0.0080	0.0030	-0.0081	-0.0100

SLOPE INDICATOR LEVEL	DEPTH TO LOAD LEVEL (in)	DEFLECTION AT SLOPE INCLINOMETER LEVEL (INCHES)							
		LOAD NUMBER 10	Deflection (in)		0.0080	LOAD NUMBER 20	Deflection (in)		0.2740
		A0	θ180	Slope	Deflection	A0	θ180	Slope	Deflection
1	0	0.0000	-0.0018	0.000110	0.1260	0.0000	-0.1004	0.001080	0.0000
2	40	-0.0180	0.0158	0.000050	0.1300	-0.0190	0.0120	0.001265	0.2170
3	80	-0.0038	0.0042	0.000000	0.0707	-0.0040	0.0060	0.001150	0.073
4	124	-0.0180	0.0180	0.000080	0.0581	-0.0120	0.0124	0.001005	0.1200
5	160	-0.0007	0.0000	0.000050	0.0079	-0.0003	0.0002	0.000005	0.0000
6	200	0.0037	-0.0034	0.000736	0.0080	0.0047	-0.0051	0.000730	0.0458
7	240	-0.0036	0.0050	0.000178	0.0107	-0.0080	0.0047	0.000730	0.3007
8	280	0.0030	0.0000	0.000100	0.0000	-0.0030	0.0030	0.000005	0.0149
9	320	-0.0050	0.0000	0.000112	0.0000	0.0000	0.0000	0.000000	0.0080
10	360	-0.0001	0.0050	0.000000	-0.1000	0.0000	0.0002	0.000000	-0.1000
11	400	-0.0001	0.1000	0.000000	-0.0010	0.000	0.0000	0.000000	-0.3018
12	439	-0.1001	0.0070	-0.000000	0.0010	-0.0084	0.0000	-0.000030	-0.3070

Table C.18 Continued

FILE No: T 8
DATE 11-Jun-96
OPERATOR: Post Deflection

ROOSEVELT BRIDGE PIER 18 NORTH
UNIVERSITY OF FLORIDA
Digitilt Indicator – Sinco 2000

PAGE No 8
Pile Group

SLOPE INDICATOR LEVEL	DEPTH TO LOAD LEVEL (in)	DEFLECTION AT SLOPE INCLINOMETER LEVEL (INCHES)							
		LOAD NUMBER 38 Deflection (in) 0.8885				LOAD NUMBER 39 Deflection (in) 0.7810			
		A0	A180	Slope	Deflection	A0	A180	Slope	Deflection
1	0	0.0004	−0.3900	1.000983	0.4740	0.0073	−0.0987	0.000918	0.7298
2	40	−0.0063	0.0061	0.001900	0.3992	−0.0071	0.0072	0.000592	0.3098
3	80	−0.0920	0.0017	0.001900	0.3189	−0.0080	0.0100	0.001905	0.8600
4	104	−0.0112	0.0100	0.001867	0.2471	−0.0007	0.0000	0.000592	0.2770
5	160	0.0067	−0.0913	0.001530	1.1797	0.0048	−0.0030	0.000500	0.2791
6	200	0.0034	−0.0054	0.000564	0.1108	1.0000	0.0000	0.001785	0.1904
7	240	−0.3001	0.0001	0.000650	0.0001	0.0012	0.0000	0.001800	0.1300
8	290	−0.3000	0.0008	0.000400	0.0091	−0.0018	0.0000	0.000600	0.0800
9	321	−0.3000	0.0080	0.000062	0.0110	−0.3072	0.0070	0.000600	1.0046
10	360	−0.3000	0.5000	0.000100	0.2008	−0.3080	0.0000	0.000073	0.0100
11	400	−0.0081	0.0080	−0.000900	−0.3610	−0.0081	0.0000	0.000501	0.0001
12	480	−0.0081	0.0000	0.000410	0.5001	−0.0002	0.2000	1.000000	0.0013

SLOPE INDICATOR LEVEL	DEPTH TO LOAD LEVEL (in)	DEFLECTION AT SLOPE INCLINOMETER LEVEL (INCHES)							
		LOAD NUMBER 40 Deflection (in) 1.1189				LOAD NUMBER 40 Deflection (in) 1.2007			
		A0	A180	Slope	Deflection	A0	A180	Slope	Deflection
1	0	0.0004	−0.0000	0.004011	1.0000	0.0118	−0.0118	0.000113	1.0008
2	40	−0.3000	0.0001	0.004118	0.8380	−0.0000	0.0021	0.000234	0.9117
3	80	−0.0170	0.0117	0.003271	0.7846	−0.0140	0.0130	0.000000	0.8000
4	124	−0.0074	0.0000	0.001500	0.5831	−0.0000	0.0000	0.001960	0.7130
5	160	0.0000	−0.0500	0.000273	0.4110	0.0001	−0.0001	0.000000	0.0010
6	200	0.0001	−0.3000	0.000011	0.3806	0.0001	−0.0042	0.002100	0.3614
7	240	−0.0011	0.0002	0.000200	0.1012	1.0001	0.0000	0.000700	1.0000
8	280	−0.0016	0.0000	0.003300	0.1112	−0.0004	0.0000	0.001000	0.1000
9	321	−0.0000	0.0000	0.000011	0.0040	−0.0000	0.0000	0.001000	0.0001
10	360	−1.0000	0.0000	0.000600	0.020	−0.0000	0.0001	0.001001	0.0000
11	400	0.3000	0.000	0.000025	0.0000	−0.0000	0.0000	0.000000	0.0140
12	440	−0.3000	0.0075	0.000502	0.0015	−0.0000	0.0000	0.000013	0.0000

Table C.18 Continued

FILE No: T-9
DATE 13-Jun-99
OPERATOR: Pose 0.000000

ROOSEVELT BRIDGE PIER 18 NORTH
UNIVERSITY OF FLORIDA
Digitilt Inclinometer Slope 2001

PAGE No 3
Pile Group

SLOPE INDICATOR LEVEL	DEPTH TO LOAD LEVEL (in)	DEFLECTION AT SLOPE INCLINOMETER LEVEL (INCHES)							
		LOAD NUMBER 7	Deflection (in)		1.7500	LOAD NUMBER 8	Deflection (in)		0.0000
		A0	A180	Slope	Deflection	A0	A180	Slope	Deflection
1	0	0.0045	-0.0184	0.008775	1.8000	0.0180	0.0180	0.008188	2.3278
2	40	-0.3000	0.0050	0.008869	1.6350	0.0080	0.0086	0.006863	1.6880
3	80	-0.2100	0.0720	0.004700	1.700	0.0000	0.0080	0.006844	1.6151
4	124	-0.3017	0.0000	0.004700	0.9000	0.0618	-0.0014	0.008010	1.2700
5	160	0.3000	-0.3000	0.010053	0.7900	0.0130	-0.0128	0.007100	0.8871
6	200	0.2117	-0.2114	0.006218	0.8900	0.2130	-0.0152	0.006113	0.6416
7	240	0.0615	-0.0618	0.003000	0.3300	0.0040	-0.0086	0.004730	0.4460
8	280	0.0050	-0.0613	0.000075	0.1060	0.0006	-0.0400	0.000127	0.2000
9	321	-0.0000	0.0050	0.000050	0.1050	-0.0047	0.0502	1.000910	0.1300
10	360	-0.1000	0.0000	0.000050	0.0477	-0.0050	0.0050	0.000050	0.0513
11	400	-0.0000	0.0080	0.000275	0.0160	-0.0028	0.0040	0.000000	0.0241
12	438	-0.0061	0.0071	0.000000	0.0074	-0.0000	0.0071	0.000015	0.0000

SLOPE INDICATOR LEVEL	DEPTH TO LOAD LEVEL (in)	DEFLECTION AT SLOPE INCLINOMETER LEVEL (INCHES)							
		LOAD NUMBER 9	Deflection (in)		0.5080	LOAD NUMBER 10	Deflection (in)		0.3000
		A0	A180	Slope	Deflection	A0	A180	Slope	Deflection
1	0	0.0080	-0.0244	0.011800	0.9898	0.0080	-0.0062	0.000418	0.2736
2	40	0.0130	-0.0000	0.011600	0.5520	-0.0061	0.0018	0.000350	0.2360
3	80	-0.0050	0.0080	0.010500	2.0710	-0.0000	0.0080	0.000275	0.2781
4	124	0.0090	-0.0000	0.010536	1.0080	-0.0080	0.0080	0.000087	0.2770
5	160	0.0130	-0.0172	0.008200	1.2141	0.0000	-0.2001	0.000335	0.4756
6	200	0.0100	-0.0180	0.007800	0.6010	0.0081	-0.0072	0.000358	0.3000
7	240	0.0080	-0.0064	0.000875	0.5410	-0.0002	0.0001	0.000773	0.2000
8	280	0.0080	-0.0060	0.000850	0.3100	0.0001	0.0001	0.000000	0.2000
9	321	-0.0080	0.0080	0.0020	0.1545	-0.0000	0.0080	0.000410	0.1000
10	360	-0.1000	0.0007	0.001330	0.0514	-0.0050	0.0444	0.000070	0.0730
11	400	-0.3602	0.0090	0.000457	0.0021	-0.0082	0.0434	0.000075	0.0080
12	438	-0.0061	0.5074	0.000810	0.0097	0.1600	0.0070	0.000050	0.0140

Table C 19 Slope Inclinometer Readings of Pile 10

PILE No T-10
DATE 13-Jun-88
OPERATOR: Liaan

ROOSEVELT BRIDGE PIER 16 NORTH
UNIVERSITY OF [illegible]

PAGE No 1
Group Test

SLOPE INDICATOR LEVEL	DEPTH TO LOAD LEVEL (IN)	DEFLECTION AT SLOPE INCLINOMETER LEVEL (INCHES)						
		LOAD NUMBER 9A			LOAD NUMBER 18			LOAD 0
		A180	A0	Sum	A180	A0	Sum	Initial Reading
1	10	0.0270	-0.3450	0.0000	0.0080	-0.0050	0.0010	0.0[illegible]
2	61	0.0000	-0.2210	0.3400	0.0010	-0.0210	0.0000	0.3420
3	81	0.0210	-0.0000	0.0410	0.1010	-0.0000	0.0170	0.0410
4	130	0.0140	-0.0000	0.2130	0.0000	-0.0080	0.0140	0.0130
5	60	0.0270	-0.0200	0.0000	0.0200	-0.0080	0.0000	0.0080
6	200	0.0210	-0.0100	0.0270	0.0210	-0.0170	0.0000	0.0070
7	248	0.0000	0.0010	0.0000	0.0000	-0.0010	0.0000	0.0000
8	288	0.0090	-0.0000	0.0000	0.0200	-0.0000	0.0170	0.0070
9	327	0.0470	-0.0000	0.1300	0.0000	0.0001	0.0050	0.0000
10	366	0.0020	-0.0400	0.0530	0.0520	-0.0110	0.0000	0.0000
11	398	0.2140	-0.0140	0.0000	0.0140	-0.0140	0.0030	0.0000
12	440	0.0000	-1.0050	0.0000	0.0000	-0.3030	0.0000	0.0000

SLOPE INDICATOR LEVEL	DEPTH TO LOAD LEVEL (IN)	DEFLECTION AT SLOPE INCLINOMETER LEVEL (INCHES)							
		LOAD NUMBER 10	Deflection (in)		0.0062	LOAD NUMBER 11	Deflection (in)		0.2000
		A180	A0	Slope	Deflection	A180	A0	Slope	Deflection
1	10	0.0000	-0.0050	0.000000	0.0720	0.0090	-0.0080	0.001275	0.2461
2	61	0.0230	-0.0220	0.000000	0.0641	0.0070	-0.0080	0.001300	0.1610
3	81	0.0230	-0.0210	0.000500	0.0300	0.0030	-0.0020	0.000400	0.1300
4	130	0.0110	0.0000	0.000000	0.0140	0.0120	-0.0110	0.001000	0.0020
5	148	0.0000	-0.0270	0.000000	-0.0080	0.0080	-0.0000	0.000000	0.0000
6	200	0.0010	-0.0100	0.000070	-0.0040	0.0030	-0.0040	0.000070	0.0000
7	248	0.0000	-0.0010	0.000000	-0.0044	0.0020	-0.0020	0.000000	-0.0020
8	288	0.0080	-0.0000	0.000000	-0.0000	0.0030	-0.0030	0.000000	0.0000
9	327	0.0400	-0.0460	0.000000	-0.0044	0.0400	-0.0450	0.000500	-0.0044
10	366	0.0010	-0.0000	-0.000000	-0.0127	0.0210	-0.0210	-0.000250	-0.0181
11	398	0.0140	-0.0140	0.000000	0.0000	0.0100	-0.0140	0.000000	-0.0000
12	440	0.0000	-0.0000	0.020000	0.0000	0.0030	-0.0400	-0.000000	-0.0000

Table C 19 Continued...

FILE No: T 30
DATE: 10-Jun-96
OPERATOR: Uzun

ROOSEVELT BRIDGE PIER N NORTH
UNIVERSITY OF FLORIDA

PAGE No 1
File Group

SLOPE INDICATOR LEVEL	DEPTH TO LOAD LEVEL (IN)	DEFLECTION AT SLOPE INCLINOMETER LEVEL (INCHES)							
		LOAD NUMBER 39 Deflection (in) 0.0600				LOAD NUMBER 40 Deflection (in) 0.0380			
		A180	A0	Slope	Deflection	A180	A0	Slope	Deflection
1	11	1.0000	-0.0240	0.000020	0.3413	1.0080	-0.0300	0.000871	0.0388
2	21	0.0470	-0.0080	0.000000	0.4077	1.0080	-0.0080	0.000500	0.0840
3	41	1.0080	-0.2040	0.000000	0.0920	1.1100	-0.0300	0.000000	0.5364
4	60	1.0140	-0.0130	0.000000	0.2707	1.0180	-0.0100	0.000000	[illegible]
5	80	1.0070	-0.0130	0.001750	0.1010	0.0000	-0.0010	0.000000	0.0000
6	100	0.0010	-0.0600	0.000070	0.1190	0.0240	-0.0300	0.000125	0.0010
7	140	0.0080	-0.2080	0.000000	1.0080	0.0000	-0.0040	0.000000	0.0101
8	180	0.0200	0.0000	0.000250	0.0480	0.0310	-0.0000	0.000000	0.0000
9	224	1.0080	-0.0800	0.000720	0.0000	1.0000	-0.0200	0.000000	0.0107
10	264	1.0020	1.0010	0.000000	1.0000	0.0030	-0.0010	0.000000	0.0107
11	400	1.0180	-0.3540	0.000000	0.1000	0.2120	-0.3100	0.000000	0.0107
12	440	1.0080	-0.2000	0.000000	1.0000	0.0080	-0.8000	0.000000	0.0000

SLOPE INDICATOR LEVEL	DEPTH TO LOAD LEVEL (IN)	DEFLECTION AT SLOPE INCLINOMETER LEVEL (INCHES)							
		LOAD NUMBER 68 Deflection (in) 1.0082				LOAD NUMBER 48 Deflection (in) 1.0000			
		A180	A0	Slope	Deflection	A180	A0	Slope	Deflection
1	11	0.0060	-0.0380	0.000070	1.0000	1.0800	-0.0080	1.000000	0.0040
2	21	0.0000	-0.0410	0.000000	0.0000	0.0300	-0.0080	0.000800	0.0030
3	41	0.0000	-0.0380	0.000750	1.7000	1.0100	-0.0130	0.000070	1.0074
4	100	0.0100	-0.0100	0.000000	1.0104	1.0000	-0.0600	0.000250	0.0010
5	160	1.0080	-0.1040	0.000000	1.4676	0.3000	-0.0080	0.000100	0.0000
6	200	1.0040	1.0000	0.000075	1.0000	0.0000	-0.0030	0.000070	1.4377
7	240	1.0000	0.0000	1.000000	0.2000	0.0080	-0.0470	0.000000	0.0130
8	280	0.0000	0.0000	1.000000	[illegible]	0.0020	-0.0000	0.000250	0.0050
9	327	0.0000	-0.0010	1.000000	0.0000	0.0000	-0.1010	0.000000	0.1000
10	364	0.0000	-0.3000	1.000000	1.0000	1.0000	-0.0050	1.000750	0.0000
11	400	1.0100	-0.7180	1.000000	[illegible]	0.0100	-0.0100	0.000000	1.0000
12	480	0.0000	-0.0000	0.000000	0.3000	0.0000	-0.0000	0.000000	0.0000

Table C 19 Continued

FILE No 7-10
DATE 13-Jun-88
OPERATOR Lionel

ROOSEVELT BRIDGE PIER 16 NORTH
UNIVERSITY OF FLORIDA
Digitil indicator -- Sinco 6000

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Pile Group

SLOPE INDICATOR LEVEL	DEPTH TO LOAD LEVEL (IN)	DEFLECTION AT SLOPE INCLINOMETER LEVEL (INCHES)							
		LOAD NUMBER 10 Deflection (in)			2.5840	LOAD NUMBER 11 Deflection (in)			2.6070
		A180	A0	Slope	Deflection	B180	B0	Slope	Deflection
1	0	0.0880	−0.0830	0.009720	0.0000	0.0000	−0.0800	0.011875	2.5630
2	41	0.0030	−0.0080	0.009730	7274	0.0000	−0.0800	0.011000	2.4180
3	81	0.0000	−0.0000	0.009730	1.6800	0.0000	−0.0400	0.010000	1.8000
4	120	0.0000	−0.0000	0.007230	1.0874	0.0000	−0.0000	0.010050	1.3200
5	156	0.0005	−0.0000	0.006000	0.6100	0.0000	−0.0000	0.008750	1.1270
6	196	0.0000	−0.0000	0.008000	0.0000	0.0030	−0.0220	0.000000	0.7300
7	236	0.0000	−0.0000	0.000750	0.2740	0.2100	−0.0100	0.007250	0.2[illegible]
8	276	0.0000	−0.0000	0.000700	0.0000	0.0000	−1.0000	0.003730	2.0040
9	317	0.0010	−0.0020	0.00[illegible]	0.1181	0.0000	−0.0020	0.001730	0.1170
10	356	2.0040	−0.0000	0.000730	0.0000	0.0050	−0.0000	0.001000	0.0000
11	396	0.2100	−0.0100	0.000730	0.0000	0.2100	−0.0100	0.000700	0.0000
12	430	2.0040	−0.0000	0.000200	0.0000	0.0000	−0.0000	0.000000	0.0000

SLOPE INDICATOR LEVEL	DEPTH TO LOAD LEVEL (IN)	DEFLECTION AT SLOPE INCLINOMETER LEVEL (INCHES)							
		LOAD NUMBER 6B Deflection (in)			3.7510	LOAD NUMBER 12 Deflection (in)			0.2000
		B180	A0	Slope	Deflection	B 180	A0	Slope	Deflection
1	2	0.0670	−0.0000	0.015777	0.0000	0.0010	−0.0070	0.000750	0.0734
2	41	0.0080	−0.0000	0.016250	0.1132	0.0200	−0.0070	0.000750	0.0130
3	81	0.0000	−0.0000	0.016250	0.5140	0.0500	−0.0500	0.000000	0.7007
4	120	0.0000	−0.0000	0.016250	1.8627	0.0100	−0.0000	0.000750	0.0003
5	158	0.0050	−0.0000	0.014750	0.4301	0.0300	−0.0000	0.000750	0.4670
6	198	0.0420	−0.0070	0.000070	0.0000	0.0000	−0.0000	0.000000	0.0000
7	238	0.0100	−0.0100	0.000000	0.0000	0.0000	−0.0000	0.000000	0.0000
8	278	0.0070	−0.0000	0.000400	0.3340	0.0000	−0.0000	0.000000	0.1300
9	317	0.0520	−0.0000	0.000050	0.[illegible]	0.0010	−0.0010	0.001000	0.1[illegible]
10	354	1.0090	−0.0040	0.001250	0.0000	0.0000	−1.0000	0.000750	0.0737
11	396	0.0780	−0.0100	0.000050	0.0084	0.0100	−0.0100	0.000750	0.0000
12	430	0.0080	−0.0000	0.000000	0.0000	0.0000	−0.0040	0.000000	0.0000

Table C 20 Slope Inclinometer Readings of Pile 11

PILE No T-11
DATE 13-Jun-90
OPERATOR: R. Lewis

ROOSEVELT BRIDGE PIER 14 [illegible]
UNIVERSITY OF FLORIDA
Sinco Digital Indicator Serial [illegible]

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Group Test

SLOPE INDICATOR LEVEL	DEPTH TO LOAD LEVEL (in)	DEFLECTION AT SLOPE INCLINOMETER LEVEL (INCHES)						
		LOAD NUMBER 6A			LOAD NUMBER 6B			LOAD 1
		A180	A0	Sum	A180	A0	Sum	Initial Reading
1	12	−0.0070	0.0150	−0.0170	−0.0072	0.0100	−0.0172	−0.0179
2	61	0.0080	−0.0071	0.0130	0.0080	−0.0050	0.0120	0.0121
3	91	0.0150	−0.0149	0.0034	0.0090	−0.0100	0.0050	0.0050
4	120	0.0127	−0.0050	0.0030	0.0127	−0.0080	0.0032	0.0035
5	160	0.0040	−0.0050	0.0026	0.0014	−0.0060	0.0110	0.0100
6	200	0.0110	−0.0080	0.0001	0.0110	0.0080	0.0050	0.0050
7	246	0.0180	−0.017	0.0060	0.0030	0.0100	0.0000	0.0080
8	258	0.0130	0.711	0.0047	0.0130	−0.0110	0.0000	0.0080
9	287	0.0130	0.0070	0.0150	0.0130	−0.0060	0.0150	0.0150
10	324	0.2610	0.0018	0.0000	0.0018	0.2610	−0.3000	−0.0004
11	358	0.0060	−0.0060	0.0030	0.0030	−0.0070	0.0150	0.0150
12	440	0.0060	−0.0027	0.0000	0.0001	−0.3620	0.0002	0.0000

SLOPE INDICATOR LEVEL	DEPTH TO LOAD LEVEL (in)	DEFLECTION AT SLOPE INCLINOMETER LEVEL (INCHES)							
		LOAD NUMBER 11 Deflection (in) 1.230				LOAD NUMBER 10 Deflection (in) 0.0000			
		A180	A0	Slope	Deflection	A180	A0	Slope	Deflection
1	12	−1.0000	1.0000	0.000682	1.130	1.0060	1.0000	0.001850	1.3460
2	61	0.0100	−1.0000	0.000517	1.0060	0.0180	0.0100	0.001560	0.2913
3	91	0.0000	−0.0170	0.000840	1.0000	0.0020	0.0100	0.001850	0.2180
4	120	0.0130	−0.0117	0.000657	0.0430	0.0050	−0.0125	0.001010	0.1370
5	180	0.0075	−0.0080	0.000000	0.0001	0.0084	−0.0000	0.000862	0.1087
6	200	0.0150	−0.0004	0.000086	0.0000	0.0026	−0.0104	0.000080	0.0710
7	246	0.0100	−0.0175	0.000087	0.0080	0.0001	−0.0060	0.000604	0.0060
8	258	0.0124	−0.0110	0.000004	0.0000	0.0060	−0.0110	0.000064	0.0000
9	287	0.0100	−0.0000	0.000032	0.0044	0.0004	−0.0060	0.000012	0.0180
10	324	0.0010	0.0015	0.000013	0.0430	0.0017	0.0018	0.000180	0.0000
11	358	0.0120	−0.0060	0.000112	0.0000	0.0130	−0.0061	0.000110	0.0000
12	440	0.0004	−0.0000	−0.000050	−0.0004	0.0004	−0.0027	0.000050	0.0000

Table C 20 Continued

FILE No: 1 01
DATE: 10-Jun-96
OPERATOR: R. Lewis

ROOSEVELT BRIDGE PIER 10 NORTH
UNIVERSITY OF FLORIDA
Slope Digital Indicator Serial 38064

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Pile Group

SLOPE INDICATOR LEVEL	DEPTH TO LOAD LEVEL (IN)	DEFLECTION AT SLOPE INCLINOMETER LEVEL (INCHES)							
		LOAD NUMBER 39 Deflection (in)			0.8930	LOAD NUMBER 40 Deflection (in)			1.0197
		A180	A0	Slope	Deflection	A180	A0	Slope	Deflection
1	12	−0.0010	0.4082	0.000580	[illegible]	0.0000	0.3013	0.000663	0.9841
2	51	0.0052	−0.3127	0.000712	0.0000	0.2170	−0.3184	0.004087	0.7344
3	84	0.0086	−0.0016	0.002863	0.0071	0.0088	−0.0034	0.003710	0.0579
4	140	0.0071	−0.0181	0.004087	0.0000	0.0101	−0.0100	0.009680	0.0087
5	180	0.0088	−0.0064	0.001758	0.7640	0.0100	−0.0130	0.000738	0.0000
6	204	0.0145	−0.0178	0.000664	[illegible]	0.0130	−0.0004	0.000099	0.7407
7	240	0.0211	−0.0140	0.000664	1.7772	0.0020	−0.0011	0.001487	0.1570
8	295	0.0148	−0.0100	0.000800	0.0600	0.0106	−0.0004	0.000180	[illegible]
9	327	0.0150	−0.0080	0.000037	7.0190	0.0110	−0.0084	[illegible]	0.0700
10	364	0.0010	0.0013	0.000130	1.0080	0.0065	0.0030	[illegible]	0.0000
11	384	[illegible]	−0.0079	−0.000036	−0.0030	0.0170	−0.0084	0.000210	0.0000
12	445	0.0050	−0.0021	0.000530	0.0010	0.0020	−0.0030	0.000000	0.0000

SLOPE INDICATOR LEVEL	DEPTH TO LOAD LEVEL (IN)	DEFLECTION AT SLOPE INCLINOMETER LEVEL (INCHES)							
		LOAD NUMBER 38 Deflection (in)			1.3050	LOAD NUMBER 39 Deflection (in)			0.8609
		A180	A0	Slope	Deflection	A180	A0	Slope	Deflection
1	60	0.0000	−0.0041	0.000430	1.4077	0.0080	−0.0000	0.000010	1.1000
2	61	0.0012	−0.0107	0.000056	0.1730	0.0000	−0.0011	0.000030	0.6744
3	91	0.0088	−0.0072	0.000064	0.0600	0.0020	−0.0000	0.000001	0.0012
4	120	0.0015	−0.0094	0.000049	0.7546	0.0060	−0.0000	0.000030	0.2272
5	204	0.0164	−0.0129	0.004615	0.5209	0.0125	−0.0130	0.000064	0.0897
6	204	0.0181	−0.0155	0.000580	0.3189	0.0145	−0.0172	0.000064	0.4800
7	264	0.0010	−0.0215	[illegible]	0.2425	0.0230	−0.0230	[illegible]	0.0050
8	264	0.0150	−0.0167	0.001527	0.1364	0.0170	−0.0187	0.000004	0.0050
9	327	0.0082	−0.0121	0.001613	0.0600	0.0000	−0.0110	0.001304	0.1386
10	364	0.0000	−0.0004	0.000000	0.0000	0.0000	0.0000	0.000004	0.0000
11	304	0.0107	−0.0000	0.000013	0.0000	0.2110	−0.0000	0.000004	0.0049
12	445	0.0000	−0.0041	0.000100	0.0000	0.0000	−0.0000	0.000004	0.0079

Table C.20 Continued

PILE No: T-11
DATE 13-Jun-96
OPERATOR R. Lewis

ROOSEVELT BRIDGE PIER 10 NORTH
UNIVERSITY OF FLORIDA
Digitilt Indicator: Sinco 2000

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Pile Group

SLOPE INDICATOR LEVEL	DEPTH TO LOAD LEVEL (in)	DEFLECTION AT SLOPE INCLINOMETER LEVEL (INCHES)							
		LOAD NUMBER 7	Deflection (in)		0.0000	LOAD NUMBER 8	Deflection (in)		0.0000
		A180	A0	Slope	Deflection	A180	A0	Slope	Deflection
1	0	0.2127	-0.3100	0.010880	0.3361	0.2100	-0.3117	0.013738	2.2100
2	34	0.2061	-0.3062	0.009460	1.2080	0.2080	-0.3039	0.012910	2.0772
3	61	0.2372	-0.2988	0.008034	1.5645	0.2430	-0.2916	0.013510	2.1860
4	100	0.2080	-0.2904	0.008016	[illegible]	0.2080	-0.2897	0.011404	1.9880
5	150	0.2480	-0.2150	0.004738	0.2881	0.2080	-0.2044	0.008610	1.2200
6	180	0.3280	-0.2100	0.008880	0.2380	0.2080	-0.2080	0.007753	0.2080
7	200	0.2080	-0.2080	0.003317	0.2080	0.2080	-0.2080	0.006510	0.2800
8	275	0.0130	-0.2160	0.002804	0.2515	0.0100	-0.2167	0.003850	0.3050
9	317	0.0130	-0.3022	0.001707	0.1386	0.0140	-0.2130	0.002080	0.1840
10	354	0.2080	0.0010	0.009480	0.0980	0.0080	-0.2580	0.002080	0.2080
11	396	0.0130	-0.2085	0.000706	0.0800	0.0130	-0.3098	0.000380	0.2080
12	435	0.2080	-0.2800	0.000654	0.0130	0.0080	-0.3600	0.000075	0.2080

SLOPE INDICATOR LEVEL	DEPTH TO LOAD LEVEL (in)	DEFLECTION AT SLOPE INCLINOMETER LEVEL (INCHES)							
		LOAD NUMBER 9	Deflection (in)		0.3080	LOAD NUMBER 10	Deflection (in)		0.2080
		A180	A0	Slope	Deflection	A180	A0	Slope	Deflection
1	0	0.2480	-0.2088	0.017809	4.1570	-0.0013	0.2037	0.000880	0.2080
2	31	0.0808	-0.3110	0.017890	3.4621	0.0130	-0.0130	0.003080	0.0720
3	61	0.0980	-0.2880	0.013019	1.8080	0.3080	-0.0027	0.003080	0.7184
4	100	0.2080	-0.0404	0.010080	2.1050	0.0180	-0.0181	0.002080	0.2080
5	150	0.0808	-0.0017	0.010880	1.0818	0.0136	-0.0088	0.002113	1.0080
6	180	0.0119	-0.3980	0.013380	1.0174	0.0180	-0.3436	0.000918	0.4173
7	250	0.3280	-0.3067	0.008880	0.8402	0.3080	-0.0021	0.000818	0.1146
8	275	0.2210	-0.3080	0.008480	0.3739	0.0130	-0.0152	0.001084	0.2080
9	317	0.0130	-0.0130	0.002480	0.0121	0.0130	-0.0118	0.000346	0.1381
10	354	0.0508	-0.3220	0.001538	0.1330	0.2080	-0.2011	0.000133	0.0675
11	396	0.0110	-0.3101	0.000888	0.3380	0.0113	-0.3080	0.000050	0.0470
12	435	0.2080	-0.3080	0.000654	0.2130	0.2080	-0.3038	0.000400	0.2080

Table C.21 Slope Inclinometer Readings of Pile 12

PILE No. T-10
DATE 10-Jun-40
OPERATOR: Robert Hogge

ROOSEVELT BRIDGE PIER 66 NORTH
UNIVERSITY OF FLORIDA
Digital Indicator Slope 2000

PAGE No. 6
Group Test

SLOPE INDICATOR LEVEL	DEPTH TO LOAD LEVEL (in)	DEFLECTION AT SLOPE INCLINOMETER LEVEL (INCHES)						
		LOAD NUMBER 1A			LOAD NUMBER 2B			LOAD 0
		A180	A0	Sum	A180	A0	Sum	Base Reading
1	60	0.0500	−0.3201	0.0177	0.0500	−0.0095	0.0008	0.0577
2	91	0.0140	−0.0177	0.0500	0.0100	−0.0179	0.0008	0.0500
3	91	0.0500	−0.0016	0.0189	0.0500	−0.0173	0.0040	0.0500
4	130	0.0150	−0.0101	0.0014	0.0152	−0.0500	0.0012	0.0018
5	168	0.0048	−0.0053	0.0500	0.0140	−0.0505	0.0008	0.0502
6	169	0.0035	−0.0081	0.0049	0.0049	0.0502	0.0500	0.0608
7	248	0.0561	−0.0099	0.0272	0.0132	0.0140	0.0373	0.0072
8	285	0.0130	0.0129	0.0036	0.0140	0.1130	0.0576	0.0498
9	307	0.0140	0.0140	0.0588	0.0141	−0.0140	0.0506	0.0500
10	169	0.0500	−0.0005	0.0192	0.0500	−0.0500	0.0500	0.0700
11	446	0.0500	−0.0080	0.0602	0.0501	−0.0601	0.0600	0.085
12	445	0.0501	−0.081	0.1211	0.0500	−0.0500	0.1270	0.1011

SLOPE INDICATOR LEVEL	DEPTH TO LOAD LEVEL (in)	DEFLECTION AT SLOPE INCLINOMETER LEVEL (INCHES)							
		LOAD NUMBER 1B Deflection (in)			0.0500	LOAD NUMBER 2B Deflection (in)			0.0500
		A180	A0	Slope	Deflection	A180	A0	Slope	Deflection
1	72	0.0500	−0.0500	0.000500	0.0505	0.0140	−0.0117	0.000205	0.2620
2	91	0.0174	−0.0181	0.000035	0.0700	0.0500	0.0140	0.000900	0.2090
3	91	0.3074	−0.0068	0.000060	0.0501	0.0063	−0.0080	0.001900	0.1538
4	130	0.0180	−0.0168	0.000075	0.0040	0.0171	−0.0178	0.000085	0.0100
5	168	0.0500	−0.0050	0.000000	0.0187	0.0502	−0.0500	0.000735	0.0036
6	169	0.0047	0.0050	0.000400	0.0185	0.0050	−0.0508	0.000400	0.0800
7	248	0.0140	0.0140	0.000000	0.0080	0.0140	−0.0140	0.000020	0.0271
8	169	0.0015	−0.0150	0.000015	0.0034	0.0107	−0.0050	0.000217	0.0142
9	307	0.0500	−0.0150	−0.000500	0.0028	0.0140	−0.0146	0.000019	0.0049
10	169	0.0047	−0.0050	−0.000500	0.0001	0.0088	−0.0500	0.000009	0.0800
11	446	0.0500	−0.0500	0.000100	−0.0015	0.0501	−0.0500	0.000000	0.0500
12	445	0.0500	−0.0810	0.000017	0.0085	0.0500	−0.0870	0.000000	0.0087

Table C.21 Continued

FILE No: [illegible]
DATE: [illegible]
OPERATOR: Robert Griggs

ROOSEVELT BRIDGE PIER [illegible] NORTH
UNIVERSITY OF FLORIDA
Digital Indicator [illegible]

PAGE No [illegible]
Pile Group

SLOPE INDICATOR LEVEL	DEPTH TO LOAD LEVEL (ft)	DEFLECTION AT SLOPE INCLINOMETER LEVEL (INCHES)							
		LOAD NUMBER [illegible] Deflection (in) [illegible]				LOAD NUMBER [illegible] Deflection (in) [illegible]			
		A180	A0	Slope	Deflection	A180	A0	Slope	Deflection
1	12	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
2	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
3	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
4	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
5	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
6	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
7	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
8	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
9	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
10	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
11	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
12	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

SLOPE INDICATOR LEVEL	DEPTH TO LOAD LEVEL (ft)	DEFLECTION AT SLOPE INCLINOMETER LEVEL (INCHES)							
		LOAD NUMBER [illegible] Deflection (in) [illegible]				LOAD NUMBER [illegible] Deflection (in) [illegible]			
		A180	A0	Slope	Deflection	A180	A0	Slope	Deflection
1	12	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
2	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
3	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
4	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
5	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
6	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
7	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
8	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
9	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
10	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
11	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
12	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

Table C.21 Continued

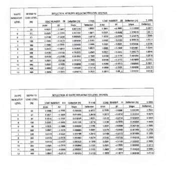

Table C.22 Slope Inclinometer Readings of Pile 16

PILE No.: T 16
DATE: 17-Jun-99
OPERATOR: David [illegible]

ROOSEVELT BRIDGE PIER 16 [illegible]
UNIVERSITY OF FLORIDA
Digitilt Indicator - Slope [illegible] / Serial [illegible]

PAGE No. 1
Group Test

SLOPE INDICATOR LEVEL	DEPTH TO LOAD LEVEL (in)	DEFLECTION AT SLOPE INCLINOMETER LEVEL (INCHES)						
		LOAD NUMBER 8A			LOAD NUMBER 8B			LOAD 0
		A180	A0	Sum	A180	A0	Sum	Initial Reading
1	12	0.0000	−0.0077	0.0148	0.0077	0.0068	0.0149	0.0148
2	61	0.0056	−0.0021	0.0070	0.0046	−0.0068	0.0074	0.0074
3	91	0.0084	−0.0046	0.0130	0.0084	−0.0051	0.0138	[illegible]
4	120	0.0000	−0.0056	0.0143	0.0000	0.0000	0.0116	0.0110
5	60	0.0113	−0.0100	0.0016	0.0104	−0.0101	0.0214	0.0214
6	600	0.0135	−0.0118	0.0049	0.0130	[illegible]	0.0040	0.0045
7	246	−0.0004	0.0014	−0.0018	−0.0001	0.0002	−0.0017	−0.0014
8	600	−0.0070	0.0000	−0.0137	−0.0075	0.0000	−0.0157	−0.0157
9	337	−0.000	0.0041	−0.0072	−0.0081	0.0000	−0.0071	−0.0170
10	364	0.0113	0.0000	0.0004	0.0015	−0.0000	0.0014	0.0004
11	600	0.0002	−0.0082	0.0174	0.0092	−0.000	[illegible]	0.0174
12	440	0.0140	−0.0138	0.0002	0.0140	0.0140	0.0002	0.0002

SLOPE INDICATOR LEVEL	DEPTH TO LOAD LEVEL (in)	DEFLECTION AT SLOPE INCLINOMETER LEVEL (INCHES)							
		LOAD NUMBER 18 Deflection (in) 0.1360				LOAD NUMBER 29 Deflection (in) 0.2691			
		A180	A0	Slope	Deflection	A180	A0	Slope	Deflection
1	10	0.0002	−0.0077	0.000000	0.000	0.0120	−0.0066	0.001305	0.0002
2	40	0.0050	−0.0030	0.000000	0.0000	0.0084	−0.0056	0.001066	[illegible]
3	91	0.0069	−0.0069	0.000000	0.0000	0.0084	−0.0073	0.001000	0.1300
4	130	0.0002	−0.0057	0.000000	0.0000	0.0096	−0.0071	0.001313	0.1181
5	180	0.0100	−0.0100	0.000075	0.0300	0.0120	−0.0112	0.000730	0.0790
6	600	0.0135	−0.0118	0.000000	0.0202	0.0140	−0.0120	0.000728	0.0487
7	240	0.0050	0.00[illegible]	0.000012	0.0118	0.0084	0.0000	0.000068	0.0080
8	600	−0.0079	0.0011	0.000100	0.0004	−0.0000	0.0079	0.000000	0.0138
9	307	−0.0001	0.0040	0.000013	0.0000	−0.0080	0.0000	0.000007	0.0008
10	364	0.0010	0.0000	0.000075	0.0000	0.0010	−0.0002	0.000000	0.0004
11	600	0.0004	−0.0000	0.000000	0.0000	0.0004	0.0004	0.000012	−0.0000
12	440	0.0140	−0.0130	0.000000	0.0016	0.0140	−0.0130	−0.000007	−0.0016

Table C.22 Continued

FILE No: T 10
DATE 11-Jun-96
OPERATOR: David Hamota

ROOSEVELT BRIDGE PIER 11 NORTH
UNIVERSITY OF FLORIDA
Digitilt Indicator - Sinco [illegible] / Serial [illegible]

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Pile Group

SLOPE INDICATOR LEVEL	DEPTH TO LOAD LEVEL (IN)	DEFLECTION AT SLOPE INCLINOMETER LEVEL (INCHES)							
		LOAD NUMBER 5B	Deflection (In)		1.4660	LOAD NUMBER 6B	Deflection (In)		1.7248
		A0 180	A0	Slope	Deflection	A180	A0	Slope	Deflection
1	10	0.2 [illegible]	-0.0112	0.000025	1.4672	0.0160	-0.0131	1.000000	1.6000
2	51	1.0000	-0.0075	0.002100	1.3000	0.0154	-0.0084	1.052100	1.5405
3	90	1.0098	-0.0002	0.000009	0.2846	0.0134	-0.0108	0.000070	1.0000
4	130	1.0099	1.0081	0.005841	0.2057	0.2110	-0.0105	0.000505	0.0010
5	168	1.1 [illegible]	-0.0102	1.025000	0.1400	0.0109	-0.0124	1.000000	0.2000
6	209	1.0100	-0.0124	0.000054	0.006	0.0102	-0.0110	0.001175	1.1000
7	248	1.0071	0.0001	1.000000	0.0017	0.0058	-0.0124	0.001013	0.0017
8	288	-0.0003	0.0074	0.000420	0.0050	-0.0050	1.0000	0.000005	0.0413
9	327	-0.0005	1.0057	1.000214	0.0058	0.0007	0.0000	1.000012	0.0191
10	366	1.0017	-0.0134	1.000470	1.0005	0.0071	0.0018	1.000020	0.0010
11	406	1.0006	-0.0134	-0.000050	-0.0054	0.0058	1.0061	-0.000001	-0.0000
12	445	0.0180	-0.0127	1.000000	-0.0010	1.0147	-0.0148	0.000012	1.0000

SLOPE INDICATOR LEVEL	DEPTH TO LOAD LEVEL (IN)	DEFLECTION AT SLOPE INCLINOMETER LEVEL (INCHES)							
		LOAD NUMBER 8B	Deflection (In)		1.0047	LOAD NUMBER 9B	Deflection (In)		1.2406
		A180	A0	Slope	Deflection	A180	A0	Slope	Deflection
1	11	0.0181	-0.0150	1.000000	1.0047	0.2100	-0.0100	0.000805	0.2721
2	51	0.0130	-0.0130	1.000013	0.8678	0.0100	-0.0180	1.000000	1.0400
3	90	0.0108	-0.0133	0.000730	0.8108	0.2102	-0.0104	0.000020	0.0010
4	90	0.0090	-0.0104	1.000500	0.4730	0.2100	-0.0144	0.000013	[illegible]
5	98	0.0100	-0.0100	1.000000	0.0015	0.0100	-0.0170	0.000000	1.0000
6	200	0.0170	-0.0100	0.001005	0.0179	1.0100	-0.0175	0.000005	1.0000
7	248	0.0030	-0.0018	0.001463	0.1184	1.0000	-0.0027	0.000005	1.1790
8	288	-0.0087	0.0030	0.000005	1.0754	-0.0003	1.0000	0.00 [illegible]	1.0000
9	327	1.0000	0.0007	0.000005	1.0004	-0.0018	1.0000	0.000087	0.0500
10	366	1.0800	-0.0010	0.000500	0.0172	1.0000	-0.0018	0.000075	0.0012
11	405	1.0000	-0.0003	1.000000	1.0040	1.0000	-0.0000	0.000100	0.0004
12	445	0.0180	-0.0100	0.000000	1.0015	1.0100	-0.0100	1.000000	0.0100

Table C.2 Continued

FILE No. 1-10
DATE 00-Jun-98
OPERATOR: David Hathelle

ROOSEVELT BRIDGE PIER 10 NORTH
UNIVERSITY OF FLORIDA
Digitilt Indicator – Sinco 50309 / Serial #9879

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Pile Group

SLOPE INDICATOR LEVEL	DEPTH TO LOAD LEVEL (IN)	DEFLECTION AT SLOPE INDICATOR LEVEL (INCHES)							
		LOAD NUMBER 7	Deflection (in)		0.2461	LOAD NUMBER 8	Deflection (in)		0.2087
		A180	A0	Slope	Deflection	A180	A0	Slope	Deflection
1	12	0.0059	−0.0019	0.007420	4.0547	0.0054	−0.0279	0.0.0000	2.2751
2	61	0.0181	−0.0177	0.006661	1.5610	0.0088	−0.0550	0.0.0100	2058
3	91	0.0194	−0.0166	0.006601	1.2756	0.0048	−0.0249	0.009815	1.4700
4	125	0.0102	−0.0177	0.006188	0.6071	0.0050	−0.0007	0.006660	3060
5	155	0.0113	−0.0050	0.006 20	0.9860	0.0050	−0.0047	0.007725	0.7515
6	200	0.0080	−0.0150	0.001750	0.3017	0.0030	−0.0050	0.000175	4.0660
7	246	0.0063	−0.0054	0.006610	0.2181	0.0008	−0.0052	0.000168	0.0656
8	300	−0.0045	0.0000	0.011605	1.0721	−0.0064	0.0049	0.000780	0.0660
9	337	−0.0016	0.0002	0.006847	0.0850	−0.0018	0.0018	0.000112	5.6720
10	364	0.0050	−0.0014	0.006660	0.0314	0.0044	−0.0018	0.006600	0.5300
11	400	0.3050	−0.0050	0.000175	0.0153	0.0000	−0.0064	0.000175	0.0160
12	446	0.0148	−0.0148	0.000110	0.0099	0.0140	−0.0142	0.000407	0.0098

SLOPE INDICATOR LEVEL	DEPTH TO LOAD LEVEL (IN)	DEFLECTION AT SLOPE INCLINOMETER LEVEL (INCHES)							
		LOAD NUMBER 12	Deflection (in)		0.1360	LOAD NUMBER 13	Deflection (in)		0.0675
		A180	A0	Slope	Deflection	A180	A0	Slope	Deflection
1	14	0.0647	−0.0200	0.003630	0.0104	0.0104	−0.0113	0.001001	0.0080
2	31	0.0060	0.0060	0.013280	0.9648	0.0086	−0.0070	0.001001	0.0099
3	61	0.0072	−0.0050	0.012770	1.0140	0.0.08	−0.0009	0.000000	0.6774
4	125	0.0050	−0.0016	0.011710	1.0660	0.0102	−0.0100	0.000210	0.0030
5	155	0.0500	−0.0050	0.000575	0.0052	0.0.02	−0.0143	0.001001	0.0500
6	205	0.0060	−0.0051	0.006600	0.0700	0.0140	−0.0153	0.001675	0.0161
7	256	0.0040	−0.0067	0.002600	0.0.60	0.0000	−0.0016	0.001400	0.0071
8	286	−0.0050	0.0084	0.001650	0.0054	0.0050	0.0050	0.000625	0.0060
9	327	−0.0013	0.0016	0.001086	0.0660	−0.0019	0.0001	0.000713	0.0606
10	366	0.0020	−0.0018	0.000600	0.0446	0.0050	−0.0014	0.000473	0.0300
11	399	0.0050	−0.0050	0.000000	0.2000	0.0050	−0.0081	0.000060	0.0100
12	449	0.015	−0.0144	0.000416	0.0064	0.0146	−0.0144	0.000467	0.0084

APPENDIX D

REACTION PILE GROUP LOAD TEST

Table D 1 Horizontal and Vertical Displacements.

LOAD NUMBER	TIME (HH:MM)	POTENTIOMETER # 1 (MS) READING (mm)	POTENTIOMETER # 1 (MS) HORIZONTAL MOVEMENT (M)	POTENTIOMETER # 2 (MW) READING (mm)	POTENTIOMETER # 2 (MW) HORIZONTAL MOVEMENT (M)	POTENTIOMETER # 3 (E) READING (mm)	POTENTIOMETER # 3 (E) HORIZONTAL MOVEMENT (M)	VERTICAL READINGS LEVEL 1 (M)	VERTICAL READINGS LEVEL 2 (M)
0 A	11:00 AM	20.000	0.000	66.000	0.000	N/A	N/A	0.00	1.20
0 B	11:08 AM								
1 A	11:12 AM	19.880	0.120	65.920	0.080	N/A	N/A	0.00	1.20
1 B	11:30 AM								
1 C	11:42 AM	19.900	0.100	65.926	0.074				
2 A	11:48 AM	19.880	0.120	65.926	0.074	N/A	N/A	0.00	1.20
2 B	11:58 AM								
2 C	12:05 PM	19.900	0.100	65.940	0.060				
3 A	12:13 PM	19.890	0.110	65.900	0.100	N/A	N/A	0.00	1.20
3 B	12:21 PM								
3 C	12:40 PM	19.800	0.200	65.900	0.100				
4 A	12:48 PM	19.740	0.260	65.880	0.120	N/A	N/A	0.00	1.20
4 B	12:58 PM								
4 C	01:10 PM	19.860	0.140	65.896	0.104				
4 D	01:24 PM								
4 E	01:30 PM								
5A	01:32 PM	19.830	0.170	65.880	0.120	N/A	N/A	0.00	1.20
5B	01:40 PM								
5C	01:58 PM	19.800	0.200	65.540	0.500				
6A	02:00 PM	19.800	0.200	65.500	0.500	N/A	N/A	0.00	1.20
6B	02:10 PM								
6C	02:20 PM	19.760	0.240	65.000	1.000				
7A	02:20 PM	19.720	0.280	67.900	2.240	N/A	N/A	0.00	1.20
7B	02:28 PM								
7C	02:50 PM	19.550	0.450	67.050	0.484				
8A	02:57 PM	19.400	0.600	66.380	0.532	N/A	N/A	0.00	1.15
8B	03:00 PM								
8C	03:15 PM	19.415	0.544	66.320	0.564				
9A	03:2? PM	19.300	0.661	66.320	0.584	N/A	N/A	0.00	1.13
9B	03:3? PM								
9C	03:44 PM	18.770	0.250	65.500	0.498				
10 A 10 B 10 C	03:30 PM	19.300	0.290	16.300	0.510	N/A	N/A	N/A	N/A
11 A 11 B 11 C	04:00 AM			N/A	N/A	2.020	0.000	N/A	N/A
12 A 12 B 12 C	04:05 PM			N/A	N/A	2.020	−0.580	N/A	N/A
13 A	04:35 PM			N/A	N/A	1.875	−1.160	N/A	N/A
13 B	04:45 PM					1.057	−1.281		
13 C									

Table D.2 Strain Gage Readings of Pile 1

PIER 3 NORTH PILE 1

LOAD #	FULL BRIDGE STRAIN GAGE READINGS							QUARTER BRIDGE STRAIN GAGE READINGS						
	91	100'	170'	260'	260	260	91E	91	100	7b	260	260	260	4.3
	με	με	με	με	με	με	με	με	με	με	με	με	με	με
0a	1	0						1	0			47	−100	
1a	1	0						1	−1			50	−105	
1b	−1	0						0	−1			30	−144	
1c	−3	−1							1			−3	−119	
2a	−2	0						1	1			30	−130	
2b	0	0						1	1			−50	130	
2c	−8	−8						1	0			5	−143	
3a	−9	−10						0	0			−38	171	
3b	−6	10						0	0			−16	−248	
3c	0	−14						0	0			−12	−268	
4a	−6	−16						0	4			−14	130	
4b	−2	12						1	0			−8	−180	
4c	−1	−30						0	0			−39	130	
4d	−1	12						4	0			−45	−194	
4e	−1	0						0	1			−31	−187	
4f	0	30						0	4			−30	−172	
5a	0	−30						0	0			1	−160	
5b	−6	81						0	0			−66	180	
5c	−5	−87						0	0			−81	−311	
6a	−6	−30						0	0			10	−307	
6b	0	−30						0	0			−30	314	
6c	−6	−36						0	0			−24	−301	
7a	−1	80						11	18			13	−212	
7b	−1	−41						11	8			−30	318	
7c	−6	−42						11	0			−81	−263	
8a	0	−43						12	11			80	−241	
8b	−3	−49						33	11			−60	−371	
8c	−8	92						14	12			−40	−223	
9a	−2	−34						13	30			30	−208	
9b	−8	−50						13	14			−17	−307	
9c	0	30						18	14			−10	−109	
10a	−4	−40						13	12			−56	881	
10b	−6	−30						0	0			−12	−307	
10c	−3	−20						0	0			−30	380	
10d	0	−10						0	0			−34	−2148	

Table D 2 Continued

PIER 13 NORTH FILE 3

LOAD	QUARTER BRIDGE STRAIN GAGE READINGS						
#	81	165	75	336	291	336	817
	με	με	με	με	με	με	με
0a	−1	0		−50			−1
1a	−2	2		−73			−2
1b	0	−1		−85			0
1c	−3	−2		82			0
2a	−7	0		34			0
2b	0	−3		341			0
2c	−1	0		204			0
3a	5	−4		236			2
3b	0	−3		178			0
3c	−3	−7		480			0
4a	−20	−3		194			4
4b	5	−3		367			0
4c	−21	−3		761			0
4d	12	−2		2580			2
4e	−5	−3		2130			0
4f	−12	−4		−1880			0
5a	−10	−2		−1080			0
5b	−16	1		−1300			4
5c	−13	−1		325			4
6a	−10	−10		−800			1
6b	−20	−10		−127			0
6c	−15	−11		−704			0
7a	−17	56		301			0
7b	−18	−56		276			4
7c	10	−13		306			0
8a	19	−14		−515			0
8b	−21	21		−380			0
8c	−21	15		−864			0
9a	−21	−53		−411			1
9b	33	−15		−400			1
9c	30	54		−1800			1
10a	10	−71		387			0
10b	18	21		−2150			0
10c	−12	−7		−2550			−8
10d	−5	−4		−1080			−4

Table D 3   Strain Gage Readings of File 5

PIER 10 NORTH   FILE 5

LOAD	FULL WIDTH STRAIN GAGE READINGS							QUARTER SPAN STRAIN GAGE READINGS						
θ	97	185	175	250	260	320	410	97	185	175	250	260	300	115
	με	με	με	με	με	με	με	με	με	με	με	με	με	με
0a				0				-0	-1	-1	0		0	-0
1a				2				1	0	0	0		0	0
1b				0				0	0	-0	-0		-0	0
1c				0				1	0	-0	-0		0	0
2a				4				0	0	1	-1		0	0
2b				0				1	1	-1	0		0	0
2c				4				0	0	-1	-0		0	0
3a				0				0	0	0	-0		-0	0
3b				0				0	1	-1	0		-0	-0
3c				7				0	0	-0	-0		-1	-2
4a				0				3	0	-2	-0		-0	-0
4b				0				5	2	-2	-0		-0	-0
4c				0				0	0	-1	-0		-0	-2
4d				0				1	0	-0	-2		-1	-1
4e				0				0	-0	0	-2		0	-0
4f				0				0	2	-0	-0		-0	-1
5a				10				0	0	0	-4		-0	-1
5b				10				0	0	-0	-4		-2	1
5c				10				0	0	-0	-0		-0	-0
6a				10				0	3	0	-0		-4	0
6b				12				0	0	-2	-0		0	-4
6c				10				0	0	-2	-0		-0	-0
7a				10				0	0	-0	0		-0	-0
7b				10				0	4	-3	-0		-0	-0
7c				10				0	0	-0	0		-0	-5
8a				10				7	0	-0	0		-0	-0
8b				10				0	0	-3	-0		-0	-0
8c				10				0	0	-3	-7		-0	-0
9a				10				0	0	0	0		-0	-0
9b				10				0	0	0	0		-7	-0
9c				10				0	0	-3	-5		0	0
10b				0				0	0	-1	-0		-0	0
11b				0				0	0	0	-1		0	-0
12b				1				0	3	1	-0		-4	0
13b				-4				0	0	0	0		-10	-0

Table D 3 Continued

PIER 11 NORTH Pt.L 1

LOAD	QUARTER BRIDGE STRAIN GAGE READINGS						
#	9T	10T	17T	21B	29B	30B	11T
	με	με	με	με	με	με	με
0a	−1		−1	0	1	1	−1
1a	−1		1	1	1	1	1
1b	−1		1	−1	1	1	1
1c	−8		1	1	1	1	−8
2a	−4		1	1	1	1	1
2b	1		1	1	1	2	−1
2c	−3		1	1	1	1	−1
3a	−1		1	1	1	1	−10
3b	1		1	1	1	6	−10
3c	−1		1	1	2	1	10
4a	−1		1	1	2	4	11
4b	−7		0	1	1	1	−11
4c	1		−1	1	1	1	−11
4d	−8		1	1	1	1	−11
4e	−2		1	1	2	4	−11
4f	7		−1	0	1	4	−11
5a	−5		−6	1	1	1	11
5b	−10		−1	0	1	6	−11
5c	−11		−1	1	1	1	−10
6a	−10		−8	−1	1	1	−11
6b	−10		−1	−1	1	0	−11
6c	−14		−1	1	−1	1	−10
7a	−11		−8	−1	1	1	−11
7b	−17		−1	−1	1	1	−11
7c	17		−6	−1	1	1	−11
8a	−17		−1	−1	1	1	11
8b	−18		−6	1	1	6	−11
8c	−20		−6	−1	0	1	12
9a	−21		1	1	1	1	−11
9b	−21		−3	−1	1	9	−11
9c	−19		−8	1	1	1	−10
10a	−11		1	1	2	1	−11
10b	−11		−4	−8	1	1	−11
10c	−5		1	−3	1	1	−11
10d	−6		−1	−8	1	1	10

Table D.4 Strain Gage Readings of Pile 6

PIER 10 NORTH PILE 1

LOAD	FULL BRIDGE STRAIN GAGE READINGS							QUARTER BRIDGE STRAIN GAGE READINGS						
P	51*	100	150	201	205	206	111**	11*	103**	111**	325	266	368	111**
	µε	µε	µε	µε	µε	µε	µε	µε	µε	µε	µε	µε	µε	µε
0a	-1	1		1			-1	1	-1	-1	-6	1	1	1
1a	1	-14		15			1	1	1	11	-1	-1	-9	0
1b	-1	-11		-10			1	1	1	15	-1	1	-1	0
1c	1	-27		-20			1	1	10	10	-1	0	-1	1
2a	10	-100		-110			20	20	10	201	-1	11	-1	1
2b	1	30		-40			1	11	20	20	-1	-4	-1	1
2c	1	-10		-50			10	11	20	20	-1	-4	-1	1
3a	-1	40		20			10	11	20	37	1	-1	-7	1
3b	11	-20		-11			1	1	12	1	-1	-17	-11	1
3c	-14	-30		-1			1	1	12	11	1	-11	-11	-1
4a	-11	-30		-4			1	-1	1	11	-4	1	-11	-9
4b	11	-30		-1			1	1	11	1	-9	-10	-1	1
4c	11	-80		-30			11	20	30	30	-1	-30	-11	-1
4d	-1	-47		-80			17	18	30	30	-8	-10	-11	1
4e	-1	-30		20			1	1	10	-1	-3	-20	-10	-1
4f	17	-20		-11			11	11	21	11	-1	11	-11	-1
5a	-30	-20		-1			1	10	11	-17	-1	-30	-14	1
5b	-30	-27		1			1	1	10	-14	-1	-20	-10	-1
5c	-30	-30		1			10	10	11	-1	-1	-10	-13	-1
6a	11	17		-8			12	11	14	-8	-9	-30	-10	-2
6b	-30	-11		-11			10	10	18	10	-1	-11	-12	-1
6c	-30	-20		11			11	1	1	-10	1	1	-10	-4
7a	-11	30		14			10	1	1	-21	-1	-14	-10	-1
7b	-41	30		1			10	12	12	1	-1	-20	-11	-1
7c	-44	-50		12			14	1	1	-1	-1	1	11	1
8a	-43	-30		11			14	1	1	-4	-3	-1	-12	-4
8b	-30	-51		11			11	1	1	-1	1	-1	-11	-1
8c	-51	50		11			10	10	1	-20	11	-11	-10	-1
9a	-30	51		26			11	1	1	-30	-1	1	-11	-1
9b	-30	-30		20			11	-1	0	-10	-3	-1	-30	-1
9c	-38	-27		30			11	1	1	-80	-1	-1	-17	-7
10a	-40	-11		30			15	1	-1	-34	-1	-10	-17	-1
10b	-30	-1		17			15	-2	-1	-30	-1	-10	-11	-1
10c	-27	-1		30			11	-2	-1	-30	-9	-1	-10	-1
10d	-10	1		14			1	-20	-1	-20	1	10	10	-1

Table D.4 Continued

PIER 10 NORTH PILE 8

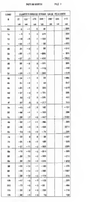
LOAD	QUARTER BRIDGE STRAIN GAGE READINGS						
#	11	13a	17	[illegible]	20	[illegible]	[illegible]
	ue	ue	ue	ue	ue	ue	ue
0a	−917	−8		−8	−1	−1	0
1a	995	−1		4	−8	−8	0
1b	897	−1		−6	2	0	−1
1c	−109	−8		1	−3	1	−1
2a	94	−1		1	−2	1	−8
2b	1087	−1		2	−2	1	0
2c	1007	−8		1	0	0	0
3a	886	−2		0	−7	1	0
3b	1030	0		0	−8	2	1
3c	999	0		−1	0	1	−8
4a	951	−8		1	−8	1	−4
4b	1001	−4		0	0	4	1
4c	888	−8		0	−1	4	1
4d	884	−8		0	−5	0	0
4e	888	−8		−1	−8	1	1
4f	880	0		0	11	0	1
5a	788	−8		0	−7	0	1
5b	796	−5		0	−8	4	0
5c	874	−8		1	−8	0	1
6a	883	−8		1	0	0	0
6b	888	−7		1	−1	0	0
6c	886	−7		0	1	0	−0
7a	888	0		0	7	0	0
7b	831	0		1	−4	7	4
7c	881	−10		0	−8	0	0
8a	880	−10		1	7	7	0
8b	−80	11		−1	−8	0	0
8c	−887	11		0	0	0	4
9a	−748	−11		0	−1	0	4
9b	−718	−10		0	−8	0	1
9c	738	−18		0	0	7	0
10a	835	−10		−1	−15	0	−0
10b	−1604	−7		−8	−11	8	4
10c	−1870	−5		−3	−18	0	0
10d	−6306	−8		−9	14	3	1

Table D.5 Strain Gage Readings of File 7

PIER MONITOR FILE 7

LOAD	FULL BRIDGE STRAIN GAGE READINGS							QUARTER BRIDGE STRAIN GAGE READINGS						
#	81	150	175	200	280	300	415	81	150	175	200	280	300	415
	ue	ue	ue	ue	ue	ue	ue	ue	ue	ue	ue	ue	ue	ue
0G		1	−98					100	0	−1			−27	−171
1a		−0	−95					66	0	0			−95	−100
1b		1	−300					−101	11	−8			−148	−421
1c		0	160					81	18	−4			−97	−389
2a		1	166					−44	18	−2			80	−390
2b		0	−274					−103	30	−14			−200	−466
2c		3	−290					−100	30	−13			−165	−417
3a		1	−270					−108	29	−14			−140	431
3b		−2	−184					48	33	13			−87	−98
3c		0	618					−30	35	−13			127	−298
4a		−0	−201					−83	48	−16			−107	143
4b		−6	−177					50	65	−30			−88	−160
4c		−3	−219					−115	62	−30			113	−309
4d		−1	−256					42	29	−14			−138	−108
4e		0	380					−107	30	−18			−107	−308
4f		−4	−381					−98	27	−15			190	320
5a		7	257					−150	60	−13			−209	−344
5b		−7	−300					−113	60	−14			−160	−271
5c		−3	−380					−157	60	−17			−300	−380
6a		−8	−380					−147	98	−60			−237	−287
6b		8	−877					−138	60	−10			−160	−229
6c		12	140					138	37	−10			96	60
7a		−12	−310					−113	60	12			−113	−3
7b		−10	−280					−96	60	56			−148	−301
7c		−15	660					210	34	−17			204	280
8a		18	300					160	60	−17			381	−347
8b		−15	−380					−150	30	12			−208	−501
8c		−17	−218					−30	60	−16			196	−147
9a		−16	818					−98	60	−10			−320	−60
9b		−14	200					−80	41	10			−154	−51
9c		−88	−380					−140	60	56			140	−18
10a		−14	−318					−80	60	−10			−111	−16
10b		−6	−336					−119	58	−13			−175	−542
10c		1	324					−100	60	−18			800	60
10d		0	−380					−100	60	−18			−134	81

Table D.5 Continued.



PIER 88 NORTH PILE 7

LOAD	QUARTER BRIDGE STRAIN GAGE READINGS						
#	21 με	100 με	175 με	200 με	290 με	305 με	415 με
0a	0	-1	0	88			300
1a	-9	1	0	270			300
1b	-18	-8	0	-100			280
1c	-30	-1	0	90			-453
2a	81	-8	0	90			-310
2b	-80	-1	0	-206			-300
2c	-87	-4	0	-430			-1043
3a	90	-8	0	-300			300
3b	81	-1	0	-118			-142
3c	-30	-1	0	342			-418
4a	-34	-1	0	98			-484
4b	81	-8	1	48			817
4c	-81	-4	0	300			-818
4d	-27	-8	0	-191			300
4e	80	-2	0	-300			401
4f	80	-8	0	-217			543
5a	-43	-8	0	182			-727
5b	-44	-4	1	-300			394
5c	90	-7	-8	-447			1002
6a	-90	-7	-1	-484			300
6b	-90	-8	0	-430			3271
6c	-58	-8	-8	-74			300
7a	90	8	8	90			-487
7b	-81	-8	-2	180			-308
7c	-87	-11	-2	-360			1309
8a	-70	-18	-2	-814			-380
8b	80	13	-2	-380			-211
8c	80	18	-2	-310			-872
9a	-90	-13	-1	-180			-374
9b	-90	-13	-2	-386			-1007
9c	90	-18	-2	-174			874
10a	-47	-8	-8	-130			300
10b	-73	-8	-5	-28			-484
10c	80	8	-8	130			-710
10d	80	-4	-8	18			100

Table D.6 Strain Gage Readings of Pile 9

PIER 10 NORTH PILE 9

LOAD	FULL BRIDGE STRAIN GAGE READINGS							QUARTER BRIDGE STRAIN GAGE READINGS						
#	81°	200°	170°	280°	295	395	410	81°	133°	150°	185	280	295	310
	uε	uε	uε	uε	uε	uε	uε	uε	uε	uε	uε	uε	uε	uε
1S	0	1	0	8		0	-0	-0	-1	0	0	-0	-0	0
1a	-4	-1	0	15		8	0	0	1	-1	1	1	0	-0
1b	8	-8	1	18		8	0	0	-1	1	-1	-1	-1	-0
1c	8	0	0	15		12	1	1	1	-0	-1	-1	-0	-1
2a	-8	-1	0	13		12	1	1	1	-1	-0	-0	-1	-0
2b	-10	0	0	10		1	1	1	-1	-0	-0	-0	-0	-0
2c	-11	1	0	10		0	1	1	-0	-0	-0	0	-0	-0
3a	-10	-4	0	10		0	0	0	-1	-0	-0	-0	0	-0
3b	-11	-1	1	10		1	0	0	1	-1	-0	-4	-0	-0
3c	-10	-1	0	15		0	0	0	0	0	-0	-0	-0	0
4a	-23	-0	0	11		1	0	0	1	0	-0	-0	-4	-0
4b	-23	-1	2	8		4	4	0	1	-0	-0	-0	-0	-0
4c	-30	0	1	10		0	1	0	1	-0	-1	-0	0	-0
4d	-30	0	0	10		1	0	0	-0	-	-0	-4	-0	-2
4e	0	10	-0	10		1	0	1	2	-0	-0	-0	-3	-1
4f	-34	-0	1	17		0	0	0	1	0	-1	-0	-4	0
5a	-61	-0	1	20		0	0	0	0	-0	-1	-0	-0	-1
5b	-61	-0	1	20		0	0	0	0	0	-0	-0	-0	0
5c	-30	-0	0	10		0	7	0	0	-0	-0	-7	0	0
6a	-38	-0	0	0		0	0	0	0	3	-0	-7	-0	-0
6b	-40	-0	0	18		1	0	1	0	-0	-1	-0	-0	-4
6c	-41	-0	0	10		0	0	0	0	-0	-0	-0	0	-0
7a	48	-10	-1	10		1	11	0	0	0	-8	-4	-0	0
7b	-40	-10	-0	10		0	10	8	4	-0	-10	-0	-0	-0
7c	-50	0	0	11		1	10	0	0	-4	-0	-0	-1	-0
8a	-54	-10	-0	10		1	10	10	0	-0	-10	-10	-0	-0
8b	00	-0	0	10		10	10	11	0	-0	-11	-10	-0	-1
8c	-30	-10	-0	10		0	14	11	0	-0	-11	-10	-0	-0
9a	-30	00	0	10		13	10	10	1	-0	-10	-11	0	-1
9b	30	-30	-0	13		10	17	10	0	-0	-10	-10	0	-1
9c	-67	-35	0	30		10	17	10	1	-0	-10	-11	-0	0
10b	-44	10	0	30		0	10	10	0	-1	-10	-0	-0	-0
11b	-40	-10	-0	0		1	10	0	0	0	-0	-0	-0	-4
12b	-30	-0	0	0		0	10	0	0	1	-0	-4	-0	-0
13b	-10	17	-0	-0		1	0	4	-1	0	-4	1	-0	-1

Table D 6   Continued.

PIER 10 NORTH   PILE 6

LOAD	QUARTER BRIDGE STRAIN GAGE READING						
#	61	130	170	210	300	380	410
	με	με	με	με	με	με	με
0a	-5	10	-0	-2	5	-1	-0
1a	-1	-0	-0	0	5	-0	-0
1b	-0	8	-1	-4	0	-2	-1
1c	-2	-0	1	-5	-1	-3	1
2a	-4	-1	-2	-8	-5	-0	1
2b	-8	0	-1	-8	-0	-0	-1
2c	-4	-0	0	-5	-0	-0	1
3a	-0	-0	-3	0	0	-4	-1
3b	8	-7	-0	-2	-2	-0	-1
3c	-0	-0	-2	-0	-1	-0	1
4a	-4	-0	0	-0	-0	-0	1
4b	8	-0	-4	0	0	-5	-1
4c	-0	-11	-4	-4	0	-6	-1
4d	-0	-0	0	0	1	-2	-5
4e	-3	-0	-2	-3	1	-0	-0
4f	-0	10	-0	-0	0	-0	1
5a	-10	-12	-0	-0	1	-5	-0
5b	-11	-10	-4	-7	1	-0	-2
5c	-11	-10	-5	-0	1	0	-2
6a	-15	-10	-0	-7	1	-5	-2
6b	-10	-10	-0	-0	0	0	-0
6c	-10	-10	-5	-0	0	0	0
7a	-10	10	0	-0	2	-0	0
7b	-10	-18	-0	-6	0	-8	1
7c	17	16	-0	-6	0	0	1
8a	-17	20	-2	-8	2	0	1
8b	-18	21	-2	-6	0	-5	1
8c	-15	-21	-8	0	2	-8	1
9a	20	-22	-6	-0	5	-0	1
9b	21	-20	-6	-2	0	-3	0
9c	21	20	-0	-3	0	-3	0
10a	10	12	-0	-6	0	-1	0
10b	-12	12	-0	-0	0	-0	0
11a	8	-11	-0	-0	0	-1	0
11b	-0	-0	0	0	1	-0	2

Table D.7 Strain Gage Readings of Pile 12.

PIER 16 NORTH PILE 12

LOAD #	FULL BRIDGE STRAIN GAGE READINGS							QUARTER BRIDGE STRAIN GAGE READINGS						
	91 UE	350 UE	175 UE	250 UE	292 UE	335 UE	410 UE	91 UE	135 UE	175 UE	195 UE	250 UE	350 UE	415* UE
0a	0	0	0	0	0	0	0	0	4	-0	0	-0	0	0
1a	-4	0	0	2	1	-1	1	1	0	1	-0	-1	-1	0
1b	-4	0	0	1	0	-1	0	1	1	0	-0	-1	-1	1
1c	0	0	-0	1	0	11	0	1	0	-0	-1	1	-1	11
2a	-8	0	0	0	0	10	1	0	0	0	-0	-1	-1	10
2b	-10	0	1	0	0	20	2	0	1	-0	-1	-0	-1	20
2c	11	0	-1	0	0	10	2	0	2	-0	-1	10	-1	10
3a	-10	0	-0	0	0	15	0	0	0	0	-0	-0	-1	11
3b	-10	0	-0	4	0	1	4	0	0	0	-1	-0	-0	0
3c	-11	0	-0	0	10	1	4	0	0	0	-1	-0	0	0
4a	-21	0	-0	0	11	0	0	0	0	1	-1	-0	-0	1
4b	-21	0	-2	5	11	0	2	0	0	1	-1	-0	-0	-0
4c	-20	0	-4	0	10	7	1	0	0	1	-1	-0	0	1
4d	-10	0	-4	0	0	4	0	0	2	0	-0	-0	-0	0
4e	1	0	-4	-0	0	1	0	-0	0	1	-0	-0	-0	-0
4f	-20	0	-4	0	10	41	0	0	0	1	-1	-0	-0	-10
5a	-28	0	-2	0	14	40	30	1	0	1	-1	-4	-4	-11
5b	-33	0	-2	0	20	60	10	1	4	5	1	0	-4	-16
5c	-30	0	-8	0	18	11	10	1	0	0	-0	-4	-4	60
6a	-00	0	-1	7	17	10	30	0	0	0	-0	-0	-0	60
6b	-00	0	-8	0	17	10	14	0	0	0	1	0	-0	-60
6c	-00	0	-0	0	10	10	10	0	1	0	-1	-0	0	60
7a	-42	0	8	7	21	10	10	11	0	0	0	-0	-0	-60
7b	-44	0	-11	0	21	10	10	11	0	0	1	0	-0	-70
7c	-45	0	-11	0	21	10	10	11	0	0	-0	-0	-0	-107
8a	-49	0	-11	1	31	16	20	10	11	0	-0	-0	-0	-109
8b	-00	0	-16	0	00	10	20	10	11	4	-1	-0	0	-113
8c	-30	0	-16	0	01	20	01	10	11	4	-1	-0	-0	-100
9a	-00	0	-14	7	30	10	06	16	10	0	1	-0	-0	-70
9b	-38	0	-18	0	20	10	27	14	20	0	-1	0	1	80
9c	30	0	-18	0	22	10	02	18	10	4	-1	-0	-7	-87
10b	-40	0	11	0	00	10	00	14	4	0	1	0	7	70
10A	-37	0	-10	-0	17	11	00	0	0	0	-0	-4	-4	-80
12b	-38	0	14	-4	11	1	10	0	1	1	-0	-1	-0	-90
13b	-11	0	-10	0	0	1	10	-0	-0	-1	-0	-0	-0	-100

Table D 7 Continued.

PIER 18 NORTH PILE 15

LOAD #	QUARTER BRIDGE STRAIN GAGE READINGS						
	91 ue	104 ue	175 ue	330 ue	350 ue	355 ue	510 ue
0a	0	0	0	-0	0	-0	0
1a	-1	-0	0	0	-1	0	0
1b	-1	-1	0	0	0	0	0
1c	-1	-1	-0	0	-0	-0	-0
2a	-0	-1	0	1	1	1	0
2b	-0	-2	-0	0	1		0
2c	-0	-0	-0	1	1	1	0
3a	-4	-0	0	1	0	0	0
3b	-0	-4	-1	1	0	0	1
3c	-0	-0	-0	1	0	0	1
4a	-0	-0	-0	0	0	0	1
4b	-0	-0	0	0	1	2	1
4c	-0	-0	-1	1	0	0	1
4d	-5	-0	0	-0	1	1	1
4e	-4	-4	-0	0	-1	0	0
4f	-4	-0	-1	1	5	0	1
5a	-0	-7	-1	0	0	0	0
5b	-0	-7	-1	0	0	4	2
5c	-0	-7	-0	1	0	0	0
6a	-0	-0	-1	0	0	0	0
6b	-0	0	0	2	4	0	0
6c	-10	-9	0	1	0	1	0
7a	-11	-10	-0	0	0	0	4
7b	12	-11	-3	0	0	0	4
7c	-10	-11	-0	0	0	0	1
8a	-13	-11	-0	0	0	0	0
8b	-14	-16	-0	1	0	0	0
8c	-14	-16	-0	1	0	0	0
9a	15	-16	-3	2	0	0	0
9b	-15	-14	-4	1	0	0	0
9c	-16	-14	-4	1	0	7	4
10a	14	-10	-0	-1	0	0	0
10b	-13	-11	-0	-0	0	0	4
10c	-11	-10	0	-0	0	0	0
10d	-5	0	0	0	0	-0	0

Table D.8 Slope Inclinometer Readings of Pile 1

PILE No: P 1
DATE: 15-Jun-98
OPERATOR: R. Varela

ROOSEVELT BRIDGE PIER 16 NORTH
UNIVERSITY OF FLORIDA
Digital Indicator Since 2001

PAGE No: 1
PIER 16

SLOPE INCLINOMETER LEVEL	DEPTH TO PILE CAP (IN)	DEFLECTION AT SLOPE INCLINOMETER LEVEL (INCHES)						
		LOAD NUMBER 08			LOAD NUMBER 09			LOAD 0
		A180	A0	Sum	A180	A0	Sum	Initial Reading
1	12	0.0040	-0.0041	0.0089	0.0000	-0.0086	0.0174	0.0614
2	61	0.0000	-0.0075	0.2188	0.0000	-0.0075	0.0137	0.2188
3	91	0.0030	-0.0060	0.3690	0.0000	0.0051	0.0088	0.3690
4	122	0.0058	-0.0060	0.0127	0.0000	0.0071	0.0102	0.0128
5	183	0.0100	-0.0190	0.0070	0.0140	-0.0250	0.0101	0.0013
6	208	0.0167	-0.0180	0.3886	0.0100	0.0126	0.0081	0.3886
7	240	0.0063	-0.0108	0.0197	0.0000	-0.0040	0.0105	0.0198
8	285	0.0080	-0.0098	0.0060	0.0061	-0.0067	0.0080	0.0080
9	327	0.0041	-0.0000	0.0000	0.0000	-0.0014	0.0003	0.0014
10	364	0.0041	-0.0060	0.0017	0.0000	-0.0015	0.0017	0.0017
11	406	0.0113	-0.0130	0.0000	0.0100	-0.0121	0.0030	0.0000
12	445	0.0081	-0.0000	0.0129	0.0000	-0.0084	0.0177	0.0177

SLOPE INCLINOMETER LEVEL	DEPTH TO PILE CAP (IN)	DEFLECTION AT SLOPE INCLINOMETER LEVEL (INCHES)							
		LOAD NUMBER 10	Deflection (in)		0.0077	LOAD NUMBER 25	Deflection (in)		0.0000
		A180	A0	Slope	Deflection	A180	A0	Slope	Deflection
1	12	0.0028	-0.3644	0.000000	0.0271	0.0034	-0.3698	0.000000	0.0300
2	61	0.0088	-0.3679	0.000000	0.0270	0.0040	-0.3679	0.000000	0.0040
3	91	0.0080	-0.3650	0.000000	0.0270	0.0040	-0.3698	0.000000	0.0008
4	130	0.0061	-0.3677	0.000170	0.0272	0.0080	-0.3678	0.000000	0.0001
5	160	0.0151	-0.3180	0.000180	0.0180	0.0100	-0.3180	0.000000	0.0000
6	208	0.0180	-0.3180	0.000112	0.0056	0.0176	-0.3180	0.000000	0.0008
7	240	0.0080	-0.3100	0.000000	0.0041	0.0080	-0.3164	0.000000	0.0030
8	285	0.0044	-0.3650	0.000027	0.0094	0.0080	-0.3000	0.000100	0.0092
9	327	0.0050	-0.3610	0.000015	0.0038	0.0061	-0.3619	-0.000000	0.0100
10	364	0.0050	-0.3613	0.000005	0.0068	0.0088	-0.3614	0.000092	0.0110
11	406	0.0110	-0.3100	0.000005	0.0038	0.0112	-0.3100	0.000100	0.0074
12	445	0.0060	-0.3000	0.000000	0.0019	0.0080	-0.3000	0.000000	0.0000

Table D.8 Continued

FILE No: [illegible]
DATE: 13-Jun-[illegible]
OPERATOR: R. Varick

ROOSEVELT BRIDGE PIER 16 NORTH
UNIVERSITY OF FLORIDA
Digital indicator Sinco [illegible]

PAGE No: [illegible]
PIER 16

SLOPE INDICATOR LEVEL	DEPTH TO PILE CAP (in)	DEFLECTION AT SLOPE INCLINOMETER LEVEL (INCHES)							
		LOAD NUMBER [illegible] Deflection (in)			0.1344	LOAD NUMBER [illegible] Deflection (in)			0.1500
		A+180	A0	Slope	Deflection	A+180	A0	Slope	Deflection
1	[illegible]	0.3000	−0.0080	0.000000	0.[illegible]	0.0080	−0.0080	0.000000	0.1500
2	51	0.0087	−0.0041	0.000000	0.0070	0.0075	−0.0080	0.000000	0.1481
3	91	0.0041	−0.0050	0.000000	0.0081	0.0080	−0.0080	0.000000	0.1040
4	130	0.0088	−0.0080	0.000000	0.0[illegible]	0.0[illegible]	−[illegible]	0.000000	0.1100
5	169	0.0107	−0.0172	0.000000	0.0000	0.0[illegible]	−0.0124	0.000000	0.0000
6	208	0.0172	−0.0100	0.000000	0.0000	0.0175	−0.0100	0.000000	0.0000
7	248	0.0091	−0.0107	0.000000	0.0205	0.0130	−0.0100	0.000000	0.0000
8	288	0.0048	−0.0050	0.000013	0.0221	0.0081	−0.0000	0.000037	0.0271
9	327	0.0081	−0.0013	−0.000012	0.0100	0.0000	−0.0113	−0.0000[illegible]	0.0100
10	364	0.0000	−0.0000	0.000000	0.0[illegible]	0.0000	−0.0010	0.000170	0.0140
11	405	0.0111	−0.0130	0.000130	0.0000	0.0131	−0.0[illegible]	0.000100	0.0[illegible]
12	445	0.0080	−0.0080	0.000000	0.0000	0.0080	−0.0080	0.000000	0.0000

SLOPE INDICATOR LEVEL	DEPTH TO PILE CAP (in)	DEFLECTION AT SLOPE INCLINOMETER LEVEL (INCHES)							
		LOAD NUMBER [illegible] Deflection (in)			0.2110	LOAD NUMBER [illegible] Deflection (in)			0.6310
		A+180	A0	Slope	Deflection	A+180	A0	Slope	Deflection
1	10	0.0084	−0.0080	0.000000	0.2121	0.0080	−0.0080	0.000075	0.2781
2	50	0.0071	−0.0084	0.000025	0.0000	0.0072	−0.0080	0.000025	0.0000
3	91	0.0062	−0.0040	0.000000	0.1000	0.0050	−0.0080	0.000000	0.0000
4	130	0.0076	−0.0080	0.000075	0.0000	0.0000	−0.0080	0.000125	0.2100
5	169	0.0107	−0.0100	0.000000	0.1225	0.0172	−0.0100	0.000100	0.0004
6	208	0.0121	−[illegible]	0.000717	0.0[illegible]	0.0187	−0.0100	0.000012	0.1230
7	248	0.0149	−[illegible]	0.000538	0.0071	0.0107	−0.0110	0.000786	0.0037
8	288	0.0058	−0.0080	0.000000	0.0000	0.0000	−0.0080	0.000010	0.0000
9	327	0.0000	−[illegible]	0.000025	0.0177	0.0000	−0.0010	−0.0000[illegible]	0.0000
10	364	0.0000	−0.0000	0.000075	0.0[illegible]	0.0013	−0.0000	0.000037	0.0000
11	405	0.0119	−0.0100	0.000100	0.0000	0.0110	−0.0128	0.000075	0.0126
12	445	0.0000	−0.0000	0.000000	0.00[illegible]	0.0000	−0.0080	0.000000	0.0000

Table D.8 Continued

PILE No: P-1
DATE: 15-Jun-96
OPERATOR: R. Verrek

ROOSEVELT BRIDGE PIER 18 NORTH
UNIVERSITY OF FLORIDA
Digitilt Indicator - Sinco 5000

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PIER 18

SLOPE INDICATOR LEVEL	DEPTH TO PILE CAP (ft)	DEFLECTION AT SLOPE INCLINOMETER LEVEL (INCHES)							
		LOAD NUMBER 78	Deflection (in)		0.0960	LOAD NUMBER 40	Deflection (in)		0.8000
		A180	B0	Slope	Deflection	A180	B0	Slope	Deflection
1	12	0.0000	-0.0080	0.000505	0.3680	0.0000	-0.0080	0.000806	0.4600
2	51	0.0050	-0.0080	0.000600	0.0960	0.0079	-0.0080	0.000870	0.4940
3	88	0.0000	-0.0074	0.001076	0.6100	0.0080	-0.0073	0.001250	0.4800
4	130	0.0060	-0.0080	0.001405	0.3791	0.0080	-0.0108	0.001330	0.8900
5	148	0.0178	-0.0080	0.001405	0.2000	0.0104	-0.0188	0.001720	0.0000
6	204	0.0160	-0.0004	0.000065	0.1000	0.0188	-0.0010	0.000[illegible]	0.2000
7	240	0.0110	-0.0080	0.000083	0.1081	0.0110	-0.0000	0.001033	0.1070
8	284	0.0080	-0.0071	0.000015	0.1060	0.0080	-0.0074	0.001085	0.1080
9	321	0.0000	-0.0081	0.000084	0.0800	0.0081	-0.0080	0.000813	0.1800
10	384	0.0011	-0.0080	0.000025	0.1080	0.0015	-0.0080	0.000806	0.0800
11	445	0.0114	-0.0108	0.000080	0.0150	0.0118	-0.0125	0.000070	0.0212
12	448	0.0000	-0.0080	0.000000	0.0090	0.0000	-0.0080	0.000000	0.0000

SLOPE INDICATOR LEVEL	DEPTH TO PILE CAP (ft)	DEFLECTION AT SLOPE INCLINOMETER LEVEL (INCHES)							
		LOAD NUMBER 0 8	Deflection (in)		0.5130	LOAD NUMBER 10	Deflection (in)		0.1740
		A180	B0	Slope	Deflection	A180	B0	Slope	Deflection
1	10	0.0080	-0.0080	0.000505	0.0180	0.0051	-0.0080	0.000606	0.1737
2	51	0.0070	-0.0080	0.000505	0.0907	0.0260	-0.0080	0.000671	0.1713
3	88	0.0080	-0.0405	0.001345	0.0874	0.0080	-0.0218	0.000506	0.1080
4	130	0.0060	-0.0110	0.001000	0.0060	0.0080	-0.0098	0.000606	0.1020
5	148	0.0108	-0.0003	0.001075	0.0080	0.0191	-0.0178	0.000606	0.0050
6	200	0.0095	-0.0050	0.001767	0.0104	0.0178	-0.0180	0.000818	0.1117
7	446	0.0108	-0.0108	0.001446	0.1051	0.0122	-0.0119	0.000005	0.0000
8	384	0.0002	-0.0008	0.001108	0.1040	0.0000	-0.0080	0.000003	0.0600
9	351	0.0012	-0.0007	0.000080	0.3778	0.0000	-0.0000	0.000012	0.0412
10	384	0.0012	-0.0000	0.000755	0.0017	0.0050	-0.0008	0.000670	0.0080
11	446	0.0118	-0.0108	0.000065	0.3241	0.0110	-0.0108	0.000606	0.0140
12	446	0.0080	-0.0109	0.000050	0.0100	0.0000	-0.0000	0.000100	0.0000

Table D.9 Slope Inclinometer Readings of Pile 5

PILE No. P 5
DATE 10-Jun-96
OPERATOR: Ed Petrowski

ROOSEVELT BRIDGE PIER 18 ? UNIT
UNIVERSITY OF FLORIDA

PAGE No. 1
PIER 18

SLOPE INDICATOR LEVEL	DEPTH TO PILE CAP (in)	DEFLECTION AT SLOPE INCLINOMETER LEVEL (INCHES)						
		LOAD NUMBER 18			LOAD NUMBER 19			LOAD 1
		A180	A0	Sum	A180	A0	Sum	Initial Reading
1	12	−0.0108	0.0120	−0.0002	−0.0101	0.0121	−0.0002	0.0002
2	51	−0.0152	0.0148	−0.0001	−0.0136	0.0147	−0.0002	0.0001
3	90	−0.0188	0.0185	−0.0013	−0.0184	0.0175	−0.0017	0.0008
4	129	−0.0214	0.0198	−0.0017	−0.0206	0.0188	−0.0012	0.0010
5	168	−0.0141	0.0164	−0.0078	−0.0142	0.0150	−0.0075	0.0008
6	206	−0.0033	0.0001	−0.0066	−0.0036	0.0195	−0.0602	0.0001
7	248	−0.0217	0.0010	−0.0427	−0.0011	0.0213	−0.0425	0.0000
8	286	−0.0109	0.0105	−0.0011	−0.0113	0.0100	−0.0011	0.0001
9	327	−0.0108	0.0103	−0.0018	−0.0085	0.0101	−0.0018	0.0010
10	364	−0.0142	0.0107	−0.0060	−0.0144	0.0134	−0.0075	0.0078
11	399	−0.0188	0.0100	−0.0071	−0.0180	0.0180	−0.0073	0.0072
12	440	−0.0179	0.0108	−0.0050	−0.0171	0.0156	−0.0064	0.0020

SLOPE INDICATOR LEVEL	DEPTH TO PILE CAP (in)	DEFLECTION AT SLOPE INCLINOMETER LEVEL (INCHES)							
		LOAD NUMBER 18 Deflection (in) 0.0197				LOAD NUMBER 20 Deflection (in) 0.0018			
		A180	A0	Slope	Deflection	A180	A0	Slope	Deflection
1	12	−0.0108	0.0101	0.000006	0.0197	−0.0108	0.0115	0.000006	0.0018
2	51	−0.0154	0.0148	0.000008	0.0137	−0.0108	0.0148	0.000006	0.0001
3	91	−0.0185	0.0130	0.000018	0.0142	−0.0167	0.0781	0.000000	0.0028
4	130	−0.0201	0.0106	0.000057	0.0118	−0.0180	0.0181	0.000036	0.0060
5	169	−0.0140	0.0152	0.000085	0.0676	−0.0142	0.0026	0.000712	0.0013
6	209	−0.0028	0.0006	0.000025	0.0080	−0.0287	0.0027	0.000013	0.0000
7	245	−0.0217	0.0218	0.000021	0.0000	−0.0085	0.0006	0.000055	0.0015
8	285	−0.0008	0.0105	0.000036	0.0075	−0.0114	0.0006	0.000713	0.0007
9	320	−0.0163	0.0153	0.000009	−0.0004	−0.0100	0.0186	0.000086	0.0217
10	364	−0.0143	0.0139	0.000005	−0.0000	−0.0146	0.0180	0.000100	0.0112
11	399	−0.0148	0.0105	−0.000006	−0.0013	−0.0182	0.0122	0.000177	0.0008
12	445	−0.0170	0.0105	0.000010	0.0000	−0.0178	0.0788	−0.000000	−0.0010

Table D 9 Continued

PILE No: [illegible]
DATE [illegible]-Jun-[illegible]
OPERATOR: [illegible]

ROOSEVELT BRIDGE PIER 10 NORTH
UNIVERSITY OF HOUSTON

PAGE No [illegible]
PIER 10

SLOPE INDICATOR LEVEL	DEPTH TO PILE CAP (in)	DEFLECTION AT SLOPE INCLINOMETER LEVEL (INCHES)							
		LOAD NUMBER [illegible] Deflection (in) [illegible]				LOAD NUMBER [illegible] Deflection (in) [illegible]			
		A180	Ai	Slope	Deflection	A180	Ai	Slope	Deflection
1	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
2	51	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
3	91	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
4	130	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
5	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
6	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
7	248	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
8	288	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
9	327	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
10	364	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
11	408	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
12	445	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

SLOPE INDICATOR LEVEL	DEPTH TO PILE CAP (in)	DEFLECTION AT SLOPE INCLINOMETER LEVEL (INCHES)							
		LOAD NUMBER [illegible] Deflection (in) [illegible]				LOAD NUMBER [illegible] Deflection (in) [illegible]			
		A180	Ai	Slope	Deflection	A180	Ai	Slope	Deflection
1	12	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
2	51	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
3	91	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
4	130	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
5	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
6	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
7	248	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
8	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
9	327	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
10	364	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
11	408	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
12	445	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

Table D.9 Continued

FILE No: P-5
DATE 12-Jun-98
OPERATOR Ed Putnam

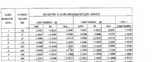
ROOSEVELT BRIDGE PIER 15 NORTH
UNIVERSITY OF FLORIDA
Digitilt Indicator - Slope 2005

PAGE No 2
PIER 15

SLOPE INDICATOR LEVEL	DEPTH TO PILE CAP (in)	DEFLECTION AT SLOPE INCLINOMETER LEVEL (INCHES)							
		LOAD NUMBER 7 Deflection (in)			0.1466	LOAD NUMBER 8 Deflection (in)			0.3245
		A180	AI	Slope	Deflection	A180	AI	Slope	Deflection
1	13	−0.0021	0.0134	−0.000025	0.1466	−0.0086	0.0713	0.000129	0.3245
2	51	−0.0130	0.0140	0.000050	0.1175	−0.0134	0.0198	0.000057	0.2188
3	51	−0.0187	[illegible]	0.000025	0.1216	−0.0188	0.0087	0.000079	0.0050
4	90	−0.0133	0.0144	0.000030	0.0871	−0.0080	0.0080	0.000015	0.1797
5	190	−0.0132	[illegible]	0.000000	0.0080	−0.0032	0.0011	0.000058	0.1427
6	406	−0.0229	0.0001	0.000000	0.0477	−0.0027	0.0086	0.000071	0.1107
7	348	−0.0087	0.0000	0.000000	0.0005	−0.0200	0.0101	0.000058	0.0801
8	406	−0.0194	0.0080	0.000050	0.0010	−0.0166	0.0080	0.000050	0.0651
9	307	−0.0136	0.0150	0.000058	−0.0013	−0.0172	0.0090	0.000055	0.0080
11	384	−0.0190	0.0134	0.000058	−0.0080	−0.0150	0.0128	0.000072	0.0010
11	406	−0.0183	0.0150	−0.000070	−0.0080	−0.0163	0.0001	0.000120	0.0102
3	406	−0.0172	0.0080	−0.000083	−0.0165	−0.0174	0.0090	0.000037	0.0001

SLOPE INDICATOR LEVEL	DEPTH TO PILE CAP (in)	DEFLECTION AT SLOPE INCLINOMETER LEVEL (INCHES)							
		LOAD NUMBER 9 Deflection (in)			0.1092	LOAD NUMBER 10 Deflection (in)			0.0001
		A180	AI	Slope	Deflection	A180	AI	Slope	Deflection
1	13	−0.0130	0.0101	−0.000050	[illegible]	−0.0086	0.0017	0.000050	0.0800
1	51	−0.0168	0.0140	0.000050	0.1092	−0.0097	0.0080	0.000018	0.0001
2	51	−0.0000	0.0081	0.000050	0.1127	−0.0165	0.0090	0.000078	0.0001
3	90	−0.0080	0.0138	0.000080	0.0010	−0.0080	0.0090	0.000237	0.2752
4	190	−0.0168	0.0115	0.0168	0.0080	−0.0111	0.0001	0.000053	0.0080
5	290	−0.0027	0.0014	0.000053	0.0001	−0.0237	0.0017	0.000001	0.0080
7	248	−0.0011	0.0100	0.000075	0.0001	−0.0219	0.0000	0.000075	0.0080
8	406	−0.0138	0.0080	0.000013	0.0001	−0.0145	0.0080	0.000080	0.0304
9	307	−0.0105	0.0144	0.000100	−0.0080	−0.0100	0.0046	0.000293	0.0021
1	384	−0.0197	0.0170	0.000025	−0.0080	−0.0164	0.0185	0.000091	0.0190
11	406	−0.0168	0.0001	−0.000025	−0.0120	−0.0144	0.0001	0.000050	0.0016
12	406	−0.0176	0.0001	−0.000100	−0.0188	−0.0174	0.0001	0.000081	0.0016

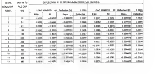
Table D.10 Slope Inclinometer Readings of Pile 6


PILE No: 6-0
DATE: 11-Jun-98
OPERATOR: Shanna/Wallace

ROOSEVELT BRIDGE PIER 14 NORTH
UNIVERSITY OF FLORIDA

PAGE No 1
PIER 14

SLOPE INDICATOR LEVEL	DEPTH TO PILE CAP (IN)	DEFLECTION AT SLOPE INCLINOMETER LEVEL (INCHES)						
		LOAD NUMBER 0A			LOAD NUMBER 0B			LOAD 0
		A180	A0	Sum	A180	A0	Sum	Initial Reading
1	12	0.0000	-0.0614	0.0047	0.0640	-0.0710	0.0027	0.1096
2	31	0.0000	-0.3048	0.0000	0.0056	0.0000	0.0030	0.1000
3	61	0.0000	-0.3000	0.0114	0.0000	-0.0840	0.0113	0.0113
4	150	0.0000	-0.0654	0.0078	0.0000	-0.0800	0.0000	0.2075
5	200	0.0000	-0.3006	0.0000	0.0000	-0.0600	0.0000	0.0000
6	205	0.0000	-0.0000	0.0100	0.0000	-0.0200	0.0130	0.0105
7	248	0.0000	-0.0620	0.0200	0.0000	-0.0554	0.0000	0.0000
8	268	0.0000	0.0000	0.0010	0.0053	0.0000	-0.0000	-0.0010
9	287	-0.0050	0.0007	-0.0000	0.0000	0.0000	0.0000	-0.0000
10	305	0.0000	-0.0001	0.0000	0.0007	-0.0000	0.0000	0.0000
11	334	0.0060	-0.0041	0.0001	0.0000	-0.0001	0.0007	0.0000
12	365	0.0074	-0.0000	0.0000	0.0073	-0.0071	0.0100	0.0104



SLOPE INDICATOR LEVEL	DEPTH TO PILE CAP (IN)	DEFLECTION AT SLOPE INCLINOMETER LEVEL (INCHES)							
		LOAD NUMBER 0A	Deflection (in)		0.0180	LOAD NUMBER 0B	Deflection (in)		0.0000
		A180	A0	Slope	Deflection	A180	A0	Slope	Deflection
1	12	0.0000	-0.0040	-0.000100	0.0187	0.0001	-0.0012	-0.000020	0.0030
2	31	0.0050	-0.0044	-0.000050	0.0080	0.0050	-0.0049	0.000020	0.0000
3	61	0.0050	-0.3000	0.000060	0.0106	0.0000	-0.0000	0.000000	0.0000
4	124	0.0040	-0.0000	0.000030	0.0106	0.0000	-0.0040	0.000037	0.0001
5	150	0.0000	0.0030	0.000125	0.0127	0.0000	-0.0048	0.000075	0.0000
6	200	0.0000	-0.0000	0.000075	0.0100	0.0050	-0.0000	0.000075	0.0000
7	248	0.0000	-0.0000	0.000000	0.0000	0.0037	-0.0000	0.000000	0.0000
8	268	-0.0000	0.0007	0.000000	-0.0013	-0.0001	0.0001	0.000000	0.0000
9	287	-0.0001	0.0000	0.000000	-0.0005	0.0000	-0.0000	0.000050	0.0000
10	305	0.0000	-0.0000	0.000050	-0.0000	0.0050	-0.0000	0.000050	-0.0000
11	306	0.0000	-0.0064	0.000000	-0.0010	0.0061	-0.0061	-0.000050	-0.0041
12	365	0.0010	-0.0071	0.000075	0.0000	0.0073	-0.0071	0.000000	0.0000

Table D 10 Continued

FILE No F-8
DATE 12-Jun-88
OPERATOR: [illegible]

ROOSEVELT BRIDGE PIER 13 NORTH
UNIVERSITY OF FLORIDA

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PIER 13

SLOPE INDICATOR LEVEL	DEPTH TO Pile CAP (in)	DEFLECTION AT SLOPE INCLINOMETER LEVEL (INCHES)							
		LOAD NUMBER 20 Deflection (in) 0.1680				LOAD NUMBER 20 Deflection (in) 0.1510			
		A180	A0	Slope	Deflection	A180	A0	Slope	Deflection
1	2	0.0000	-0.3017	-0.000015	0.1680	0.0001	-0.0017	-0.000050	0.1580
2	8	0.0062	-0.0001	0.000078	0.1660	0.0000	-0.0001	0.000005	0.1548
3	31	0.0000	-0.3000	0.000050	0.1604	0.0001	-0.0000	0.000050	0.1470
4	120	0.0046	-0.3000	0.000075	0.3000	0.0040	-0.0000	0.000005	0.1200
5	169	0.0001	-0.3000	0.000075	0.1000	0.0000	-0.0000	0.000020	0.1053
6	209	0.0006	-0.3000	0.000079	0.053	0.0000	-0.3000	0.000010	0.0800
7	240	0.0000	-0.3007	0.000050	0.0080	0.004	-0.0007	0.000005	0.0006
8	656	0.3000	0.0000	0.000075	0.0000	0.0000	0.0000	0.000050	0.0088
9	327	0.3000	0.3000	0.000050	0.0187	0.0001	0.0000	0.000030	0.0030
10	384	0.0000	-0.3000	0.000100	0.0000	0.0001	-0.0004	0.000000	0.0114
11	600	0.0046	-0.3000	0.000000	0.3620	0.0000	-0.2060	0.000003	0.0000
12	440	0.3273	-0.3073	0.000000	0.0000	0.0073	-0.0072	0.000050	0.0000

SLOPE INDICATOR LEVEL	DEPTH TO Pile CAP (in)	DEFLECTION AT SLOPE INCLINOMETER LEVEL (INCHES)							
		LOAD NUMBER 20 Deflection (in) 0.2100				LOAD NUMBER 20 Deflection (in) 0.2730			
		A180	A0	Slope	Deflection	A180	A0	Slope	Deflection
1	10	0.3000	-0.3013	-0.000005	0.4100	0.0000	-0.0013	0.000073	0.2977
2	31	0.3000	-0.3000	0.300000	0.0175	0.0000	-0.0052	0.000073	0.0000
3	61	0.3000	0.3001	0.000030	0.0082	0.0000	-0.0001	0.000050	0.0000
4	180	0.3067	-0.3000	0.000020	0.1791	0.0000	-0.0001	0.000050	0.0000
5	80	0.3000	-0.3004	0.000075	0.1487	0.0044	-0.0004	0.001220	0.1800
6	200	0.3000	-0.3000	0.000025	0.1180	0.0001	-0.0000	0.000030	0.1427
7	240	0.3000	-0.3000	0.000000	0.0021	0.0000	-0.0000	0.000000	0.1000
8	600	0.0000	0.3000	0.000025	0.0081	0.0000	-0.0000	0.000053	0.0000
9	327	0.3000	0.0000	0.000050	0.0086	0.0000	-0.0001	0.000000	0.0020
10	384	0.3000	-0.3000	0.000000	0.0180	0.0001	-0.0004	0.000030	0.0006
11	600	0.3040	0.3064	0.000000	0.0070	0.0073	-0.0040	0.000080	0.0159
12	440	0.3073	-0.3073	0.000050	0.0070	0.0074	-0.0073	0.000180	0.0000

Table D.10 Continued

FILE No. 7-6
DATE 18-Jun-86
OPERATORS Bhyme/Hernse

ROOSEVELT BRIDGE PIER 14 NORTH
UNIVERSITY OF FLORIDA
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PAGE No. 6
PIER 14

SLOPE INDICATOR LEVEL	DEPTH TO PILE CAP (in)	DEFLECTION AT SLOPE INDICATOR LEVEL (INCHES)							
		LOAD NUMBER 19 Deflection (in)			0.3604	LOAD NUMBER 10 Deflection (in)			0.4160
		Δ (in)	Δθ	Slope	Deflection	Δ (in)	Δθ	Slope	Deflection
1	18	0.0036	0.0037	−0.000675	0.3500	0.0021	−0.0017	−1.000330	0.4173
2	61	0.0067	−0.0046	0.004079	0.3500	0.0050	−0.0046	−0.000001	0.4160
3	91	0.0071	−0.0046	0.001300	0.3406	0.0000	−0.0000	0.001150	0.0000
4	130	0.0011	−0.0046	0.001050	0.2972	0.0000	−0.0091	0.001479	0.3543
5	183	0.0046	0.0000	0.001501	0.2400	0.0006	−0.0046	0.001520	0.0000
6	206	0.0006	0.0001	0.001079	0.1900	0.0000	−0.0046	0.000330	0.2962
7	248	0.0041	−0.0042	0.001180	0.1540	0.0000	−0.0046	0.000950	0.1947
8	288	0.0006	0.0046	0.001023	0.1060	0.0000	−0.0000	0.001179	0.1270
9	327	0.0046	−0.0001	0.000330	0.0500	0.0007	−0.0000	0.000830	0.0607
10	364	0.0021	0.0007	0.000830	0.0300	0.0001	−0.0000	0.000830	0.0472
11	398	0.0046	0.0000	0.000330	0.0177	0.0000	−0.0046	0.000330	0.0196
12	446	0.0006	−0.0073	0.000830	0.0000	0.0016	−0.0075	0.000330	0.0000

SLOPE INDICATOR LEVEL	DEPTH TO PILE CAP (in)	DEFLECTION AT SLOPE INDICATOR LEVEL (INCHES)							
		LOAD NUMBER 0.0 Deflection (in)			0.8000	LOAD NUMBER 18 Deflection (in)			0.1047
		Δ (in)	Δθ	Slope	Deflection	Δ (in)	Δθ	Slope	Deflection
1	12	0.0000	0.0010	−0.000075	0.0007	0.0020	−0.0010	−0.000330	0.1350
2	61	0.0000	0.0046	0.000975	0.4601	0.0016	−0.0046	0.000035	0.1300
3	91	0.0075	0.0007	0.001300	0.0046	0.0001	−0.0046	0.000090	0.0011
4	130	0.0006	0.0052	0.000879	0.4194	0.0046	0.0000	0.000879	0.1790
5	183	0.0046	0.0044	0.001779	0.3484	0.0000	−0.0046	0.004879	0.1300
6	206	0.0070	−0.0091	0.001079	0.2790	0.0000	−0.0046	0.000033	0.1300
7	248	0.0006	−0.0046	0.000046	0.3175	0.0007	−0.0046	0.000830	0.1181
8	288	0.0018	−0.0071	0.001830	0.1300	0.0000	0.0000	0.000730	0.0800
9	327	0.0010	0.0046	0.000040	0.0800	0.0006	−0.0001	0.000700	0.0900
10	364	0.0012	0.0011	0.000700	0.0071	0.0010	0.0000	0.000500	0.0010
11	398	0.0046	−0.0046	0.000330	0.1600	0.0046	−0.0046	0.000330	0.0090
12	446	0.0070	−0.0070	0.000330	0.0180	0.0006	−0.0074	0.000700	0.0010

Table D 11 Slope Inclinometer Readings of Pile 7

PILE No. 0-7
DATE 13-Jun-00
OPERATOR: Dupree/Horace

ROOSEVELT BRIDGE PIER 08 NORTH
UNIVERSITY OF FLORIDA

PAGE No 0
PIER 18

SLOPE INCLINOMETER LEVEL	DEPTH TO PILE CAP (IN)	DEFLECTION AT SLOPE INCLINOMETER LEVEL (INCHES)						
		LOAD NUMBER 0A			LOAD NUMBER 0B			LOAD 0
		A180	A0	Sum	A180	A0	Sum	Initial Reading
1	12	−0.3000	0.0087	−0.0112	−0.3058	0.0080	−0.3100	−0.0118
2	31	−0.3000	0.0082	−0.0181	−0.0072	0.0078	−0.3152	−0.0081
3	91	−0.0010	0.0032	−0.0000	0.0000	0.0081	−0.0000	−0.0080
4	120	0.0000	0.0021	−0.0000	−0.5000	0.0018	−0.0000	−0.0087
5	180	0.0044	0.0058	−0.0130	0.0000	0.0008	−0.0120	−0.0101
6	208	0.0000	0.0008	0.0002	−0.0000	0.0008	−0.0000	−0.0004
7	248	−0.0000	0.0000	−0.0080	−0.0020	0.0008	−0.0000	−0.0007
8	488	0.0043	0.0080	−0.0125	−0.0000	0.0008	−0.0030	−0.0104
9	387	−0.0007	0.0000	−0.0000	−0.0080	0.0018	−0.0008	−0.0027
10	488	0.0008	0.0080	−0.0084	−0.0000	0.0008	−0.0080	−0.2081
11	480	0.3008	0.0101	−0.0180	−0.2081	0.0088	−0.0180	−0.0094
12	443	0.0124	0.0180	−0.0000	−0.0120	0.0134	−0.0000	−0.0001

SLOPE INCLINOMETER LEVEL	DEPTH TO PILE CAP (IN)	DEFLECTION AT SLOPE INCLINOMETER LEVEL (INCHES)							
		LOAD NUMBER 10		Deflection (in)	0.0077	LOAD NUMBER 00		Deflection (in)	0.0000
		A180	A0	Slope	Deflection	A180	A0	Slope	Deflection
1	12	−0.3001	0.0080	0.000076	0.0074	−0.3000	0.0080	0.000405	0.0012
2	31	0.0034	0.0070	0.000073	0.0066	−0.3074	0.0078	0.000075	0.0014
3	91	−0.0000	0.0004	0.000405	0.0430	−0.3001	0.0080	0.000001	0.0080
4	000	0.001	0.0013	0.000007	0.0018	−0.3018	0.0014	0.000005	0.0077
5	120	−0.3047	0.0000	0.000250	0.0290	−0.3000	0.0000	0.000001	0.0000
6	200	0.2000	0.0001	0.000075	0.0072	0.0000	−0.0001	0.000001	0.0100
7	244	0.0000	0.0000	0.000100	0.0128	−0.0020	0.0008	0.000050	0.0000
8	000	0.0000	0.0000	0.000072	0.0000	−0.0044	0.0007	0.000025	0.0012
9	387	0.0011	0.0007	−0.000075	0.0000	−0.0002	0.0018	0.000071	−0.0000
10	480	0.0000	0.0001	0.000000	0.0000	−0.0040	0.0080	−0.000000	−0.0000
11	480	−0.3000	0.0004	0.000070	0.0000	0.0001	0.0080	−0.000100	−0.0000
12	440	−0.0100	0.0100	−0.000000	−0.0000	−0.0120	0.0131	−0.000000	−0.0000

Table D 11 Continued

FILE No: [illegible]
DATE 12-Jun-[illegible]
OPERATOR: [illegible]

ROOSEVELT BRIDGE PIER 10 NORTH
UNIVERSITY OF FLORIDA

PAGE No [illegible]
PIER 10

SLOPE INDICATOR LEVEL	DEPTH TO PILE CAP (IN)	DEFLECTION AT SLOPE INCLINOMETER LEVEL (INCHES)							
		LOAD [illegible]		Deflection (in)	[illegible]	LOAD [illegible]		Deflection (in)	[illegible]
		A180	A0	Slope	Deflection	A180	A0	Slope	Deflection
1	12	-0.0057	0.0060	0.000025	[illegible]	-0.0066	0.0060	0.000075	[illegible]
2	53	-0.0071	0.0075	0.000075	0.0030	-0.0072	0.0074	0.000075	0.1346
3	91	-0.0020	0.0027	0.000050	0.0000	-0.0029	0.0028	0.000000	[illegible]
4	130	-0.0060	[illegible]	0.000400	[illegible]	-0.0067	0.0060	0.000500	0.1161
5	[illegible]	-0.0065	0.0061	0.000400	0.0000	-0.0060	0.0040	0.000000	0.0000
6	208	0.0060	-0.0061	0.000425	0.0000	0.0061	-0.0060	0.000075	0.2729
7	246	-0.0058	0.0060	[illegible]	0.0000	-0.0000	[illegible]	0.000000	0.0000
8	286	-0.0047	0.0061	0.000075	[illegible]	-0.0080	0.0050	0.000075	0.0000
9	[illegible]	-0.0000	0.0018	0.000075	0.0000	-0.0017	0.0000	0.000000	[illegible]
10	[illegible]	-0.0080	0.0080	0.000700	0.0000	-0.0010	0.0040	[illegible]	[illegible]
11	[illegible]	-0.0080	0.0088	0.000075	0.0000	-0.0080	0.0088	[illegible]	-0.0080
12	442	-0.0130	0.0131	-0.000000	[illegible]	-0.0130	0.0134	-0.000130	[illegible]

SLOPE INDICATOR LEVEL	DEPTH TO PILE CAP (IN)	DEFLECTION AT SLOPE INCLINOMETER LEVEL (INCHES)							
		LOAD [illegible]		Deflection (in)	[illegible]	LOAD [illegible]		Deflection (in)	[illegible]
		A180	A0	Slope	Deflection	A180	A0	Slope	Deflection
1	12	-0.0060	0.0060	[illegible]	[illegible]	-0.0060	0.0060	0.000400	[illegible]
2	[illegible]	-0.0060	0.0060	[illegible]	0.0000	-0.0077	0.0077	[illegible]	0.0000
3	91	-0.0018	0.0020	0.000400	0.1308	-0.0017	0.0023	0.000400	[illegible]
4	130	-0.0000	0.0000	[illegible]	0.1732	-0.0008	0.0000	[illegible]	[illegible]
5	[illegible]	0.0060	0.0061	0.001000	[illegible]	0.0000	0.0060	[illegible]	0.2000
6	[illegible]	0.0030	0.0030	0.000075	[illegible]	[illegible]	-0.0000	[illegible]	[illegible]
7	246	-0.0018	0.0020	[illegible]	[illegible]	-0.0022	0.0023	[illegible]	0.0000
8	[illegible]	-0.0060	0.0000	[illegible]	0.0001	-0.0040	0.0040	[illegible]	0.0000
9	[illegible]	-0.0007	0.0011	[illegible]	[illegible]	-0.0088	0.0012	[illegible]	0.0492
10	364	-0.0000	0.0000	[illegible]	[illegible]	-0.0030	0.0030	[illegible]	0.0000
11	[illegible]	-0.0000	0.0064	0.000073	0.0000	-0.0080	0.0080	[illegible]	[illegible]
12	442	-0.0100	[illegible]	[illegible]	0.0000	-0.0030	0.0030	0.000100	0.0000

Table D.11 Continued

FILE No P 7
DATE 13-Jun-96
OPERATOR [illegible]

ROOSEVELT BRIDGE PIER 13 NORTH
UNIVERSITY OF FLORIDA

PAGE No 6
PIER 13

SLOPE INDICATOR LEVEL	DEPTH TO PILE CAP (IN)	DEFLECTION AT SLOPE INCLINOMETER LEVEL (INCHES)							
		LOAD NUMBER 20	Deflection (in)		0.3467	LOAD NUMBER 40	Deflection (in)		0.4270
		A180	A0	Slope	Deflection	A180	A0	Slope	Deflection
1	12	-0.0058	0.0068	0.000173	0.3800	-0.0068	0.0068	0.000826	0.6097
2	51	-0.0068	0.0075	0.000623	0.3017	-0.0085	0.0076	0.000825	0.4100
3	91	-0.0015	0.0068	0.000600	0.3119	-0.0011	0.0068	0.000860	0.3017
4	130	-0.0061	0.0068	0.001130	0.2760	0.0068	-0.0000	0.001000	0.0000
5	169	-0.0058	0.0047	0.000800	0.0287	0.0088	0.0058	0.001000	0.2973
6	209	0.0062	-0.0043	0.001273	0.8680	0.3011	-0.0070	0.001069	0.4000
7	248	-0.0068	0.0068	0.000068	0.1388	-0.0015	0.0068	0.000800	0.1100
8	288	-0.0058	0.0068	0.000605	0.0068	-0.0058	0.0068	0.001120	0.1150
9	327	-0.0004	0.0007	0.000805	0.0611	-0.0102	0.0066	0.000500	0.0000
10	368	-0.0058	0.0068	0.000500	0.0344	-0.0688	0.0058	0.000800	0.0000
11	406	-0.0088	0.0068	0.000273	0.0167	-0.0062	0.0088	0.000825	0.0010
12	445	-0.0130	0.0130	0.000000	0.0058	-0.0130	0.0129	0.000000	0.0097

SLOPE INDICATOR LEVEL	DEPTH TO PILE CAP (IN)	DEFLECTION AT SLOPE INCLINOMETER LEVEL (INCHES)							
		LOAD NUMBER 9 0	Deflection (in)		0.5002	LOAD NUMBER 10	Deflection (in)		0.1610
		A180	A0	Slope	Deflection	A180	A0	Slope	Deflection
1	20	-0.0004	0.0068	0.000675	0.4802	-0.0080	0.0088	-0.000580	0.1610
2	51	-0.0088	0.0075	0.000675	0.4813	-0.0073	0.0068	0.000125	0.1340
3	91	-0.0015	0.0017	0.000130	0.4842	-0.0088	0.0088	-0.000130	0.1200
4	130	0.0068	0.0032	0.001000	0.4764	-0.0084	0.0017	0.000000	0.1080
5	169	-0.0030	0.0054	0.000800	0.3444	-0.0068	0.0087	0.000880	0.1000
6	209	0.0017	-0.0014	0.001725	0.2730	0.0088	-0.0080	0.000800	0.1024
7	248	-0.0013	0.0027	0.001060	0.2077	-0.0038	0.0068	0.000800	0.0817
8	288	-0.0004	0.0068	0.000800	0.1461	-0.0088	0.0088	0.000895	0.0080
9	327	0.0068	0.0037	0.000800	0.0868	-0.0067	0.0068	0.000805	0.0060
10	364	-0.0058	0.0088	0.000730	0.0268	-0.0037	0.0088	0.000106	0.0297
11	406	-0.0088	0.0068	0.000500	0.0068	-0.0088	0.0062	0.000078	0.0096
12	445	-0.0130	0.0127	0.000500	0.0118	-0.0129	0.0128	0.000085	0.0006

Table D.12 Slope Inclinometer Readings of Pile 9

PILE No: 9 - 9
DATE: 13-Jun-96
OPERATOR: [illegible]

ROOSEVELT BRIDGE PIER 14 NORTH
UNIVERSITY OF FLORIDA
[illegible]

PAGE No: 9
PIER 14

SLOPE INDICATOR LEVEL	DEPTH TO PILE CAP (IN)	DEFLECTION AT SLOPE INCLINOMETER LEVEL (INCHES)						
		LOAD NUMBER 3A			LOAD NUMBER 4B			LOAD 0
		A180	A0	Sum	A180	A0	Sum	Initial Reading
1	12	-0.0001	0.0372	-2.0050	-2.0080	0.0623	-0.2721	-0.2783
2	51	-0.0000	0.0063	-0.3860	-0.3360	0.0058	-2.0084	-0.3860
3	91	-0.3016	0.0500	-0.9464	-0.3013	0.0058	-0.4800	-0.3811
4	130	-0.0050	0.0048	-0.0471	-0.0050	0.0500	-0.0470	-0.0471
5	139	-0.0000	0.0329	-2.0050	-0.0290	0.0058	-0.0650	-0.0863
6	209	0.0062	0.0014	-0.3860	-0.0060	0.0015	-2.0001	-0.3860
7	248	0.0000	0.0063	-0.0560	-0.0050	0.0058	-2.0050	-1.0500
8	288	-0.0061	0.0060	-0.0757	-0.0060	0.0042	-0.3360	-0.0737
9	327	-0.3630	0.0000	-2.0000	-2.0000	0.0000	-0.3000	-0.0800
10	366	-0.0060	0.0320	-1.0001	-2.0050	0.0036	-0.3818	-2.0050
11	406	-0.5070	0.0062	-0.3050	-0.3075	0.0300	-0.3075	-2.0630
12	445	-0.0000	0.0068	-0.0718	-0.3360	0.0068	-0.3018	-0.0050

SLOPE INDICATOR LEVEL	DEPTH TO PILE CAP (IN)	DEFLECTION AT SLOPE INCLINOMETER LEVEL (INCHES)							
		LOAD NUMBER 3A	Deflection (in)		0.0004	LOAD NUMBER 4B	Deflection (in)		0.0057
		A0	A180	Slope	Deflection	A0	A180	Slope	Deflection
1	12	-0.0066	0.0500	0.000000	0.0064	-0.3050	0.0500	-0.000000	0.0058
2	51	-0.3016	0.0501	0.000130	0.0060	-0.3013	0.0500	0.000175	0.0074
3	91	0.0000	0.0016	0.000030	0.0060	-0.0016	0.0032	0.000130	0.0063
4	130	-0.0000	0.0500	0.000057	0.0076	-0.0060	0.0036	0.000115	0.0000
5	139	-0.0004	0.0016	0.000000	0.0068	-0.3060	0.0079	0.000000	0.0000
6	209	-1.0000	0.0014	0.000063	0.0058	-0.3081	0.0000	0.000100	0.0153
7	248	-0.3000	0.0000	0.000030	0.2107	-0.4007	0.0000	0.000000	0.0004
8	288	-0.3060	0.0000	0.000010	0.2148	-0.3071	0.0000	0.000000	-0.0006
9	327	-0.3000	0.0002	0.000120	0.2140	-0.3471	0.0000	0.000000	-0.0010
10	366	-2.0000	0.0050	0.000120	0.7000	-0.3010	0.0000	0.000000	-2.0000
11	406	-3.0000	0.0000	0.000000	0.7000	-0.4000	0.0000	-0.000000	-0.0004
12	445	-0.0000	0.0007	0.000000	0.0003	-0.0000	0.0000	-0.000040	-0.0000

Table D 12 Continued

PILE No: P-6
DATE: [illegible]
OPERATOR: [illegible]

ROOSEVELT BRIDGE PIER 16 NORTH
UNIVERSITY OF FLORIDA
[illegible]

PAGE No: 2
PIER 16

SLOPE INDICATOR LEVEL	DEPTH TO PILE CAP (IN)	DEFLECTION AT SLOPE INCLINOMETER LEVEL (INCHES)							
		LOAD NUMBER [illegible] Deflection (in) [illegible]				LOAD NUMBER [illegible] Deflection (in) 0.1300			
		A180	A0	Slope	Deflection	A180	A0	Slope	Deflection
1	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	0.1300
2	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
3	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
4	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
5	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
6	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
7	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
8	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
9	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
10	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
11	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
12	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

SLOPE INDICATOR LEVEL	DEPTH TO PILE CAP (IN)	DEFLECTION AT SLOPE INCLINOMETER LEVEL (INCHES)							
		LOAD NUMBER [illegible] Deflection (in) [illegible]				LOAD NUMBER [illegible] Deflection (in) [illegible]			
		A180	A0	Slope	Deflection	A180	A0	Slope	Deflection
1	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
2	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
3	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
4	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
5	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
6	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
7	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
8	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
9	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
10	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
11	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
12	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

Table D 12 Continued

PILE No: P-4
DATE 13-Jun-00
DESCRIPTION: Statnamic

ROOSEVELT BRIDGE PIER 14 NORTH
UNIVERSITY OF FLORIDA
Serial 2005

PAGE No: 4
PIER 14

SLOPE INDICATOR LEVEL	DEPTH TO PILE CAP (IN)	DEFLECTION AT SLOPE INCLINOMETER LEVEL (INCHES)							
		LOAD NUMBER 10 Deflection (in) 0.3581				LOAD NUMBER 09 Deflection (in) 0.4346			
		A180	A0	Slope	Deflection	A180	A0	Slope	Deflection
1	13	−0.0048	0.0060	0.000418	0.3580	−0.0095	0.0090	0.000600	0.4326
2	61	−0.0050	0.0020	0.000606	0.3530	−0.0081	0.0100	0.000650	0.4262
3	91	−0.0184	0.0210	0.000650	0.3337	−0.0190	0.0011	0.001170	0.4000
4	135	−0.0186	0.0205	0.000960	0.2905	−0.0184	0.0215	0.001537	0.3900
5	180	−0.0274	0.0000	0.001425	0.2410	−0.0258	0.0000	0.000800	0.2956
6	208	−0.0250	0.0005	0.001000	0.1670	−0.0061	0.0080	0.001580	0.0010
7	240	−0.0000	0.0080	[illegible]	0.1980	−0.0201	0.0081	0.001270	0.1700
8	288	−0.0095	0.0070	0.000634	0.0000	−0.0000	0.0008	0.001012	0.1201
9	327	0.0457	0.0470	0.000607	0.0000	−0.0454	0.0474	0.000775	0.0000
10	364	−0.0195	0.0500	0.000600	0.0000	−0.0287	0.0110	0.000588	0.0487
11	406	0.0271	0.0500	0.000537	0.0011	−0.0070	0.0036	0.000047	0.0271
12	442	−0.0046	0.0300	0.000501	0.0000	−0.0095	0.0000	0.000600	0.0110

SLOPE INDICATOR LEVEL	DEPTH TO PILE CAP (IN)	DEFLECTION AT SLOPE INCLINOMETER LEVEL (INCHES)							
		LOAD NUMBER 44 Deflection (in) 0.5605				LOAD NUMBER 11 Deflection (in) 0.1799			
		A180	A0	Slope	Deflection	A180	A0	Slope	Deflection
1	13	−0.0005	0.0005	0.000500	0.5605	−0.0005	0.0074	0.000040	0.1799
2	61	−0.0000	0.0010	0.000600	0.4941	−0.0005	0.0006	0.000550	0.1720
3	91	−0.0167	0.0007	0.000600	0.4975	−0.0008	0.0030	0.000500	0.1700
4	132	−0.0186	0.0011	0.001797	0.4014	−0.0013	0.0041	0.000500	0.1600
5	180	−0.0005	0.0005	0.001800	0.3400	−0.0008	0.0010	0.000500	0.1520
6	208	−0.0056	0.0071	0.001700	0.2070	0.0070	0.0006	0.000500	0.1100
7	240	−0.0258	0.0271	0.001800	0.1980	−0.0070	0.0085	0.000500	0.0990
8	280	−0.0006	0.0005	0.001212	0.1876	−0.0050	0.0081	0.000518	0.0720
9	327	−0.0452	0.0471	0.000670	0.0626	−0.0005	0.0005	0.000500	0.0507
10	364	−0.0095	0.0017	0.000600	0.0096	−0.0005	0.0006	0.000500	0.0096
11	406	−0.0005	0.0006	0.000417	0.0076	−0.0040	0.0040	0.000518	0.0005
12	480	−0.0005	0.0005	0.000501	0.0120	−1.014	0.0036	0.000500	0.0075

Table D.13 Slope Inclinometer Readings of Pile 12

FILE No: P-12
DATE: 11-Jun-99
OPERATOR: Marty

ROOSEVELT BRIDGE PIER 18 NORTH
UNIVERSITY OF FLORIDA
Model 50302

PAGE No 1
PIER 18

SLOPE INDICATOR LEVEL	DEPTH TO PILE CAP (IN)	DEFLECTION AT SLOPE INCLINOMETER LEVEL (INCHES)						
		LOAD NUMBER 0A			LOAD NUMBER 00			LOAD 1
		A180	A0	Sum	A180	A0	Sum	Initial Reading
1	14	0.0079	-0.0277	0.0047	0.0081	-0.0078	0.0000	0.0000
2	51	-0.0021	0.0001	-0.0005	-0.0029	0.0000	-0.0002	-0.0005
3	91	0.0008	-0.0008	0.0014	0.0010	-0.0008	0.0013	0.0005
4	130	0.0101	-0.0100	0.0001	0.0101	-0.0101	0.0004	0.0004
5	160	0.0012	-0.0018	0.0030	0.0000	-0.0012	0.0041	0.0000
6	200	0.0000	-0.0221	0.0598	0.0000	0.0000	0.0000	0.0000
7	240	0.0000	-0.0050	0.0005	0.0000	-0.0050	0.0000	0.0001
8	285	0.0000	-0.0000	0.0137	0.0011	-0.0008	0.0000	0.0008
9	327	0.0184	-0.0180	0.0014	0.0107	-0.0107	0.7004	0.0016
10	360	0.0030	-0.0000	0.0008	0.0000	-0.0000	0.0058	0.0160
11	399	0.0000	-0.0000	0.0700	0.0000	-0.0000	0.0772	0.0759
12	440	0.0271	-0.0277	0.0030	0.0000	-0.0000	0.0038	0.0100

SLOPE INDICATOR LEVEL	DEPTH TO PILE CAP (IN)	DEFLECTION AT SLOPE INCLINOMETER LEVEL (INCHES)							
		LOAD NUMBER 10	Deflection (in)		0.0000	LOAD NUMBER 20	Deflection (in)		0.0000
		A 180	A0	Slope	Deflection	A180	A0	Slope	Deflection
1	14	0.0000	-0.0010	0.000013	0.0136	0.0079	-0.0298	0.000613	0.0028
2	51	-0.0008	0.0008	-0.000071	0.0109	-0.0050	0.0000	0.000004	0.0006
3	91	0.0017	-0.0011	0.000036	0.0107	0.0014	-0.0018	0.000003	0.0000
4	130	0.0107	-0.0104	0.000066	0.0000	0.0130	-0.0114	0.000012	0.0006
5	160	0.0021	-0.0014	0.000060	-0.0014	0.0008	-0.0008	0.000008	0.0006
6	200	0.0000	-0.0053	0.000000	-0.0008	0.0001	-0.0201	0.000000	0.0100
7	240	0.0000	-0.0000	-0.000075	-0.0156	0.0017	-0.0000	0.000075	0.0000
8	280	0.0071	-0.0000	0.000008	-0.0126	0.0070	-0.0000	0.000008	-0.0001
9	320	0.0000	-0.0107	0.000000	-0.0126	0.0008	-0.0110	0.000000	-0.0001
10	360	0.0000	-0.0008	0.000025	-0.0147	0.0008	-0.0008	0.000000	-0.0137
11	400	0.0000	-0.0000	-0.000037	-0.0000	0.0003	-0.0000	-0.000037	-0.0137
12	440	0.0000	-0.0000	-0.000000	-0.0000	0.0071	-0.0071	-0.000016	-0.0130

Table D 13 Continued.

FILE No: P-13
DATE 13-Jun-95
OPERATOR Marty

ROOSEVELT BRIDGE PIER 16 NORTH
UNIVERSITY OF FLORIDA
Model 50309

PAGE No 1
PIER 16

SLOPE INDICATOR LEVEL	DEPTH TO PILE CAP (in)	DEFLECTION AT SLOPE INCLINOMETER LEVEL (INCHES)							
		LOAD NUMBER 30	Deflection (in)		1.3828	LOAD NUMBER 40	Deflection (in)		1.1650
		A180	A0	Slope	Deflection	A180	A0	Slope	Deflection
1	12	1.0279	-0.0072	0.000110	1.3784	1.0005	-0.3284	0.000006	1.1137
2	51	-0.0091	1.0031	0.000006	1.3000	1.0028	1.0000	0.000029	1.1724
3	91	1.0013	1.0017	1.0000 3	1.0969	5.0009	-0.1820	0.000736	1.1144
4	130	1.0190	-0.1181	0.00041	1.0000	1.0190	-0.1120	0.000000	1.1304
5	169	1.0027	-0.1000	0.000000	1.0000	1.0040	1.0042	0.000 3	1.0098
6	208	1.0089	-0.0084	0.000475	1.0349	1.0088	-0.1880	0.000000	0.0604
7	248	1.0006	-0.0004	0.000081	1.0145	1.0000	-0.0211	0.000001	1.0487
8	288	1.0072	-0.1000	0.000037	1.0004	1.0075	-0.0024	0.000000	0.0061
9	327	0.0127	-0.0145	0.000000	-0.0004	1.0173	1.0180	0.000544	0.0007
10	366	1.0198	-0.1001	0.000075	-0.0178	0.0000	-0.0004	0.000475	-0.0004
11	406	1.0000	-0.1000	-0.000137	-0.0141	5.0009	-0.1038	-0.000000	-0.0116
12	445	1.0279	-0.0072	-0.000037	-0.0088	1.0074	-0.1080	-0.000281	-0.0088

SLOPE INDICATOR LEVEL	DEPTH TO PILE CAP (in)	DEFLECTION AT SLOPE INCLINOMETER LEVEL (INCHES)							
		LOAD NUMBER 50	Deflection (in)		0.3085	LOAD NUMBER 60	Deflection (in)		0.0320
		A180	A0	Slope	Deflection	A180	A0	Slope	Deflection
1	12	1.0279	-0.0034	0.000112	1.3049	1.0077	-0.0278	0.000047	0.3028
2	51	-0.0020	1.0020	1.000004	1.0000	-0.0000	1.0000	0.000029	0.2800
3	91	1.0004	-0.0000	1.000000	1.1000	1.0010	-0.1821	0.000806	0.2400
4	130	1.0180	-0.0001	1.0000 1	1.1040	1.1100	-0.1200	0.000067	0.2070
5	169	1.0020	-0.0090	1.0000 1	1.1000	1.0087	-0.0041	0.000000	0.1000
6	208	1.0080	-0.1000	0.000075	1.1200	1.0068	-0.1000	0.001 75	0.1304
7	248	1.0015	-0.1011	1.000 75	1.0625	1.0218	-0.1000	1.000000	0.0680
8	288	1.0077	-0.0007	1.000060	1.0300	1.0006	-0.1000	0.000000	0.0508
9	327	1.0130	-0.0180	1.000000	1.0087	1.0001	-0.0172	0.000775	0.0600
10	366	1.0000	-0.1000	1.000000	1.0000	1.0000	-0.0007	1.000000	0.0080
11	406	1.0050	-0.0054	-0.000000	-0.0165	1.0050	-0.0000	-0.000000	-0.0106
12	445	1.0080	-0.0074	-0.000013	-0.0084	1.0080	-0.0074	-0.000097	-0.0084

Table D 13 Continued

FILE No: P-00
DATE 13-Jun-00
OPERATOR Marty

ROOSEVELT BRIDGE PIER 16 3 DATE
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Model 50000

PAGE No 1
PIER 16

SLOPE INDICATOR LEVEL	DEPTH TO PILE CAP (IN)	DEFLECTION AT SLOPE INCLINOMETER LEVEL (INCHES)							
		LOAD NUMBER 7B Deflection (in)			0.0000	LOAD NUMBER 9B Deflection (in)			1.0000
		A180	A0	Slope	Deflection	A180	A0	Slope	Deflection
1	13	0.0000	−0.0002	0.000998	0.0481	0.0070	−0.0061	0.000743	0.0088
2	61	−0.4000	0.0000	0.000594	1.0302	−0.0005	0.0020	0.000000	0.0000
3	91	0.0000	−0.0003	0.001986	1.0017	0.0001	−0.0007	0.001263	0.3019
4	130	0.0000	−0.0000	0.001218	0.0000	0.0000	−0.0018	0.001387	0.0307
5	180	0.0000	−0.0047	0.001218	1.2000	1.0047	0.0000	0.001680	0.2770
6	209	1.0000	0.1012	0.001275	1.1400	1.0078	−0.0013	0.001550	0.2180
7	258	0.0000	−0.0029	0.001005	1.1253	1.0008	−0.0030	0.001105	0.1000
8	305	1.3000	−0.0001	0.0008 3	1.0000	1.0000	−0.0000	0.001 100	0.1001
9	337	0.0177	−0.0170	0.000000	1.0001	1.0174	−0.0100	0.001075	0.0000
10	384	1.0007	−0.0010	0.000000	1.0037	1.0000	−0.0010	0.000000	0.0001
11	406	0.0002	−0.0000	0.000000	−0.0000	0.0002	−0.0006	0.001368	−0.0000
12	445	0.0400	−0.0070	−0.000000	−1.0004	0.0070	0.0071	−0.00023	−1.0044

SLOPE INDICATOR LEVEL	DEPTH TO PILE CAP (IN)	DEFLECTION AT SLOPE INCLINOMETER LEVEL (INCHES)							
		LOAD NUMBER 8-8 Deflection (in)			0.0000	LOAD NUMBER 12 Deflection (in)			0.1004
		A180	A0	Slope	Deflection	A180	A0	Slope	Deflection
1	12	0.0079	−0.0001	0.000013	0.4007	0.0077	−0.0000	0.000057	0.1040
2	51	−0.0000	0.0000	0.000000	0.4074	−0.0005	0.0071	−0.000000	0.1400
3	91	0.0000	−0.0000	0.001368	0.0000	0.0010	−0.0013	0.000 34	0.1171
4	130	0.0040	−1.0000	0.001610	0.4100	0.0100	−0.0001	0.000000	0.1117
5	180	0.0000	−0.0002	0.001710	0.3464	0.0005	−1.0000	0.000013	1.1014
6	208	0.0000	−0.0008	0.001360	0.2020	0.0000	−1.0000	1.000070	0.0001
7	260	1.0000	−0.0000	0.001300	0.2077	0.0011	−0.00 0	0.000000	0.0000
8	305	0.0000	−0.0000	0.001360	0.1001	0.0075	−0.0000	0.000001	0.0000
9	337	0.0102	−0.0100	0.001360	0.0010	0.0100	−0.0154	1.000070	1.0000
10	384	0.3017	−0.0010	1.000000	0.0000	0.0007	−0.0000	1.000000	1.0000
11	406	0.0000	−0.0002	0.001360	0.0108	0.0000	−0.0000	−0.000000	−0.0000
12	445	0.0374	−0.0000	0.001360	0.0000	0.0071	−0.0077	−1.000100	−0.0000

APPENDIX E

P-Y CURVES

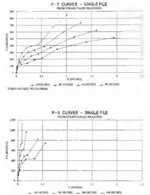


Figure E.1 P-y Curves Based on Strain Gage Measurements for the Single Pile

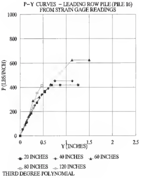


Figure E 2 P-y Curves Based on Strain Gage Measurements for a Leading Row Pile (Test Pile Group)

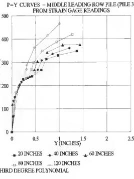


Figure E 3 P-y Curves Based on Strain Gage Measurements for a Middle Leading Row Pile (Test Pile Group)

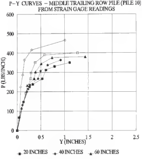


Figure E.4 P-y Curves Based on Strain Gage Measurements for a Middle Trailing Row Pile (Test Pile Group)

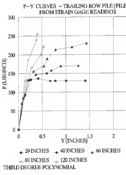
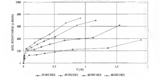
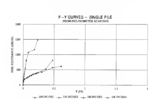


Figure E 5 P-y Curves Based on Strain Gage Measurements for a Trailing Row Pile (Test Pile Group).



ROOSEVELT BRIDGE



ROOSEVELT BRIDGE

Figure E 6 P-y Curves Based on Inclinometer Readings for the Single Pile

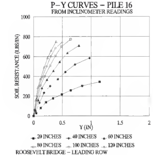


Figure E-7 P-y Curves Based on Inclinometer Readings for a Leading Row Pile (Test Pile Group).

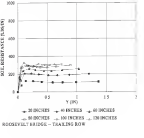
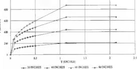
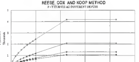


Figure E 8   P-y Curves Based on Inclinometer Readings for a Trailing Row Pile (Test Pile Group)

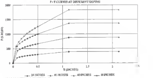


ANGLE OF INTERNAL FRICTION 30
k = 60 pci



ANGLE OF INTERNAL FRICTION 30 and 41
k = 60 and 125 pci

Figure E.9 P-y Curves Using Reese et al Method (Pre-Driving In Situ Tests).



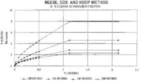
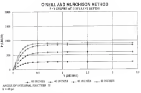


Figure E.10 P-y Curves Using Reese et al Method (Post-Driving In Situ Tests)



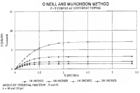


Figure E 11 P y Curves Using O'Neill and Murchison Method (Pre-Driving In Situ Tests)

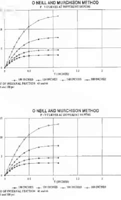


Figure E 12 P-y Curves Using O'Neill and Murchison Method (Post-Driving In Situ Tests)

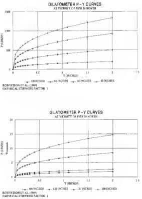
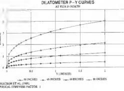


Figure E 13 P-y Curves Using DMT Robertson et al. Method (Pre-Driving In Situ Tests)



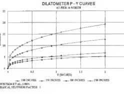


Figure E 14 P-y Curves Using DMT Robertson et al. Method (Post-Driving In Situ Tests)

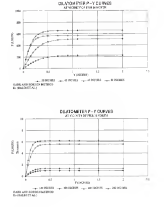


Figure E-15 P-y Curves Using DMT Gabr and Borden Method (Pre-Driving In Situ Tests).

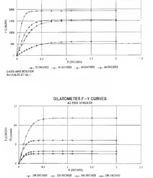
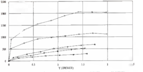


Figure E.16 P-y Curves Using DMT Gabr and Borden Method (Post-Driving In Situ Tests)



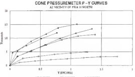


Figure E.17 P-y Curves Using PMT Robertson et al. Method (Pre-Driving In Situ Tests)

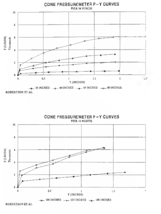


Figure E 18 P-y Curves Using PMT Robertson et al Method (Post-Driving In Situ Tests)

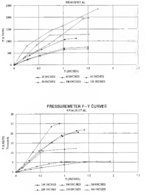


Figure E 13 P-y Curves Using PMT Briaud et al Method (Pre-Driving In Situ Tests).

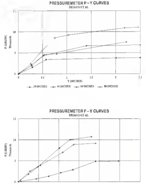
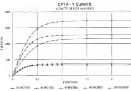


Figure E.20 P-y Curves Using PMT Briaud et al. Method (Post-Driving In Situ Tests)



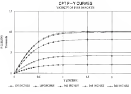
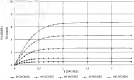


Figure E 21 P-y Curves Generated from CPT Correlation (Pre-Driving In Situ Tests)



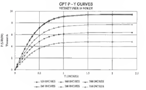


Figure E 22 P-y Curves Generated from CPT Correlation (Post-Driving In Situ Tests)

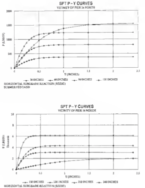


Figure E.23 P-y Curves Generated from SPT Correlation
(Pre-Driving In Situ Tests)

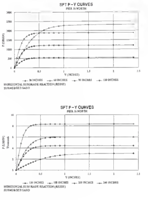


Figure E.24 P-y Curves Generated from SPT Correlation (Post-Driving In Situ Tests)

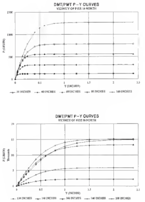


Figure E 25 P-y Curves Generated from DMT/PMT Correlation (Pre-Driving In Situ Tests)

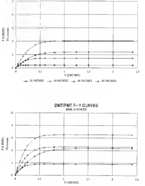


Figure E.26 P-y Curves Generated from DMT/PMT Correlation (Post-Driving In Situ Tests)

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The author was born November 8, 1961, in La Plata, Buenos Aires, Argentina, where his father was studying agricultural engineering. The next year his family moved back to Lima, Peru. He attended Santa Rosa High School in Lima and graduated in 1978. Then, he entered the National University of Engineering in April, 1979, and received his bachelor's degree in civil engineering in 1984. He earned the Title of "Ingeniero Civil" in 1985 after defending his thesis on "Interaction between bending moments and axial load in shear walls." He then worked in the Geotechnical Laboratory of the Peruvian-Japanese Center of Seismic Investigation and Disasters, until comming to the USA concentrating his studies in the field of geotechnical engineering.

Mr. Ruesta received his Master of Engineering degree at the University of Florida in 1992. He worked on a project, "Load Test Database on Piles," under the supervision of Dr. J. L. Davidson. Then, he continued his studies at UF toward the Ph.D. degree. He plans to return to his country and work as a geotechnical engineer.

I certify that I have read this study and that in my opinion it conforms to acceptable standards of scholarly presentation and is fully adequate, in scope and quality, as a dissertation for the degree of Doctor of Philosophy.

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Professor of Civil Engineering

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